SOLUTIA

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October 24, 2011

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Reference: RCRA Interim Measures – Groundwater Modeling Report

Solutia Site; 1 Monsanto Road, Nitro, West Virginia
EPA ID. No. WVD039990965

Dear Tom and Bill,

Per the request of the EPA and the WV DEP, please find attached a report entitled *Groundwater Model Development and Flow Simulations, Solutia Nitro Site, Solutia, WV.* This report details the development of the groundwater model and the resulting findings regarding groundwater flow, water level changes, potential groundwater pumping needs and Kanawha River effects following the installation of the slurry wall elements of the approved interim measures for the site. GSI Environmental located in Houston, Texas was selected to do the modeling as Solutia has worked with them on similar efforts and found them to be highly expert in this field.

If you have any questions, please call me at (314) 674-6717 or I can be reached via e-mail at mlhous1@solutia.com.

Sincerely,

Michael L. House

Michael L. House

Manager, Remedial Projects
Solutia Inc.
575 Maryville Centre Drive.
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cc: Ron Potesta, Mike Light – Potesta & Associates

GROUNDWATER MODEL DEVELOPMENT AND FLOW SIMULATIONS

SOLUTIA NITRO SITE
NITRO, WEST VIRGINIA

9 September 2011

Prepared for:

Solutia Inc.
St. Louis, Missouri

GSI ENVIRONMENTAL

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Issued: 9 September 2011

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ENVIRONMENTAL

Executive Summary

As requested by Solutia Inc. a model was constructed to simulate groundwater flow in the area of the Solutia Nitro Site in Nitro, West Virginia (Figure 1). The site is the location of former chemical manufacturing facilities and disposal areas, and groundwater containing constituents of concern is discharging into the Kanawha River adjacent to the site. Interim measures consisting of slurry walls around three source areas and cap and cover systems are planned to control the surface water recharge and infiltration to groundwater, minimizing transport to the river.

Hydrogeologic Conditions

Hydrogeologic site data used in the construction of the groundwater flow model was taken from site reports provided by Solutia. Groundwater at the Nitro site is present in a narrow alluvial aquifer, aligned parallel to the river channel, that extends from the ground surface to depths of 30 to 60 feet. The upper part of the alluvial aquifer, designated "Zone A", is significantly less permeable than the lower part, designated "Zone B." Zone A extends to a depth of approximately 30 to 45 feet, with the water table typically at a depth 20 to 30 ft below the ground surface. For the purpose of the modeling program, horizontal hydraulic conductivities for Zones A and B were estimated based on the geometric mean of hydraulic conductivities determined from slug tests implemented across the site in these two units. The slug tests indicate that the hydraulic conductivity of Zone A is approximately 0.5 ft/d. Zone B extends approximately 10 to 20 feet from the bottom of Zone A to an elevation of approximately 535 ft mean sea level (msl), where sandstone and shale bedrock is encountered. The hydraulic conductivity of Zone B based on the slug tests is approximately 7 ft /d. Head measurements indicate that there is an upward hydraulic gradient from groundwater in the bedrock to Zone B.

Three pumping tests were conducted in the bedrock to determine the hydraulic conductivity of the bedrock formation. The extraction wells at all three pumping test locations went dry after a short period, which precluded conventional pumping test analysis methods. Instead, the pumping test data was analyzed by treating the pumping well locations as variable head boundaries in three idealized MODFLOW models. The values of horizontal (Kx and Ky) and vertical (Kz) hydraulic conductivities estimated from the MODFLOW simulations were as follows:

Unit	Kx/Ky (ft/d)	Kz (ft/d)
Zone A	NA	0.1
Zone B	NA	0.35
Bedrock	0.51 to 0.86	0.0012 – 0.035

Groundwater Flow Model Set-up and Calibration

The groundwater flow model consists of three layers. The top layer, Layer 1, corresponds to the Zone A, Layer 2 corresponds to Zone B, and Layer 3 corresponds to an upper portion of the bedrock. Surface elevations were established from a digital elevation model. The divide

- Simulations indicate that a total pumping rate of 2.4 gpm is needed to maintain an inward hydraulic gradient at the source areas following installation of barrier walls that exhibit a hydraulic conductivity of 3×10^{-8} cm/s. The number of wells and pumping rates determined for each unit are:

Site Area	Number of Wells	Total Flow Rate (gpm)
WTA	3	0.5
PDA	4	1.2
PA	2	0.7
Total	**9**	**2.4**

- The simulations indicate that the inward hydraulic gradient can be maintained with pumping only in Zone A. Four existing Zone A wells can be utilized in the pumping system. These pumping rates are probably conservatively high because the hydraulic conductivity of the bedrock could be significantly lower than the value determined from pumping tests and model calibration.

- A rise in the Kanawha River stage of 2 feet is unlikely to have a significant impact on site groundwater flow patterns. A river stage increase of 5 feet would increase hydraulic heads in wells at the site and move the river/Armour Creek groundwater divide further west. A lowering of the Kanawha River stage would lower hydraulic heads across the site but would not impact groundwater flow patterns significantly.

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GROUNDWATER MODEL DEVELOPMENT AND FLOW SIMULATIONS

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1.0 Introduction

1.1 Background

The Solutia Nitro site is located on the east bank of the Kanawha River approximately 15 miles west-northwest of Charleston in Putnam County, southwestern West Virginia (Figure 1). The site, formerly known as Flexsys America L.P. (Flexsys), is a former chemical manufacturing plant that produced various chemical compounds in the early 1910s through mid-2004.

Past site investigations have determined that shallow groundwater containing site constituents is migrating from several site source areas and is discharging into the Kanawha River. Potesta and Associates, Inc. (Potesta) has prepared an interim measures work plan that specifies barrier walls around portions of three source areas: i) the Past Disposal Area (PDA), ii) Process Area (PA), and iii) Waste Treatment Area (WTA) to reduce constituent migration in groundwater. Solutia Inc. (Solutia) has retained GSI Environmental Inc. (GSI) to prepare a groundwater flow model to assist in determining the effect of the barrier walls on groundwater flow patterns and the groundwater monitoring well network.

1.2 Project Objectives

The specific objectives of the groundwater flow modeling project are to:

- Determine the effect of the proposed barrier walls on groundwater flow patterns and elevations under the selected wall configurations, and specifically, determine if barrier wall construction would cause flooding upgradient of the source areas.
- Determine the effect of altered groundwater flow patterns on the existing monitoring well network, and specifically:
    - Determine if altered groundwater flow patterns following barrier wall construction could adversely affect groundwater that is not currently impacted.
    - Determine the optimum locations for monitoring well pairs to evaluate the performance of the interim measure by monitoring the hydraulic head difference across the barrier walls.
- Estimate the pumping rate within each barrier wall needed to maintain an inward hydraulic gradient.
- Confirm that changes in the Kanawha River stage will not have a significant impact on groundwater flow patterns.

This report describes the conceptual model for the groundwater flow model, groundwater flow model construction and calibration, and the results of the simulations performed to meet the project objectives.

2.0 Site Description

The description of the site in this section is taken directly from the interim measures work plan prepared by Potesta (2010) and the reports of Roux and Associates Inc. (1995, 1999), which were provided by Solutia to GSI.

2.1 Site History

Chemical production at the Nitro site began at the site in 1918 when the United States Government started producing smokeless powder (nitrocellulose) for use in World War I. The site was purchased by the Rubber Services Company in 1921 and used for the manufacturing of chloride, phosphate and phenol compounds. Monsanto Company (Old Monsanto) purchased the facility in 1929 from Rubber Services Company, expanding operations to include the production of the herbicide 2,4,5-trichlorophenoxyacetic acid (2,4,5-T) and sodium trichlorophenoxyacetic acid. A byproduct of 2,4,5-T production is 2,3,7,8 tetrachlorodibenzo-para-dioxin (TCDD), which has been detected in surface soils at the Nitro Site.

Production of the herbicide continued until 1969. Several of the units associated with the production of the herbicide were decontaminated, demolished and buried on site during the early 1970s. Activities began during the second quarter of 2004 to dismantle, decontaminate, and remove all surface structures including the wastewater treatment plant facility. Demolition was completed in December 2005.

2.2 Regional Geology and Physical Site Setting

The Nitro site is located within the Allegheny Plateau physiographic province in the southwestern part of the state of West Virginia. The topography in the area surrounding the site is typical of the hills and valleys of the maturely dissected Allegheny Plateau. The Kanawha River and its tributaries form an intricate dendritic drainage pattern, and the area contains numerous deep valleys separated by narrow ridges. Topographic relief in the area is several hundred feet, and only a relatively small portion of the land area is flat. Hilltops within several miles of the site rise to elevations of approximately 1,200 feet above sea level. The lowest elevations in the area are along the Kanawha River at about 560 feet above sea level.

Flat land occurs mainly along stream valleys where it forms alluvial terraces, or flood plains. A prominent alluvial terrace has been developed along the Kanawha River which extends from upstream of the City of Charleston, a distance of over seventy miles downstream to the confluence of the Kanawha River with the Ohio River. The alluvial terrace consists of relatively flat land bordering the river and averages about 4,000 feet in width in the vicinity of the site. The surface elevation of the alluvial terraces decreases downstream from an elevation of approximately 600 feet at Charleston to approximately 580 feet at Nitro. The Kanawha River has incised into the alluvial terrace and meanders back and forth between the valley walls. The level of the Kanawha River is typically 20 to 30 feet below the level of the surface of the alluvial terrace.

The alluvial terraces along the Kanawha River are underlain by unconsolidated alluvial deposits consisting predominantly of sand, silt and clay with minor gravel. The upper part of the alluvial deposits typically contains fine-grained silt and clay. Coarse sand and gravel are often found in the lower alluvial deposits near the bedrock interface. The alluvial deposits are reported to be laterally variable over short distances due to the lenticular nature of individual beds. Published geologic reports indicate the thickness of the alluvial deposits ranges from 30 to 60 feet in the vicinity of Nitro due to variations in the depth to bedrock.

Bedrock in the immediate vicinity of the site consists of sedimentary rocks of the Conemaugh Group of Pennsylvanian age. This geologic unit contains an interbedded sequence of sandstone, shale and mudstone with thin beds of limestone and coal. The beds are near horizontal or gently inclined, and bedding dips generally less than 5 degrees to the northwest.

Published reports cited in reports provided by Solutia indicate that in many places saline groundwater is encountered in consolidated bedrock 100 to 300 feet below the elevation of the major streams. Locally, saline water also occurs in shallow aquifers, due to the upward migration of groundwater along zones of higher permeability. These conditions are reportedly due to the general upward vertical difference in hydraulic head in the valley bottoms, which causes a regional upward component of groundwater low in the valleys and an up welling of salt brines from great depths.

2.3 Kanawha River Characteristics

The site is located in the lower part of the Kanawha River Basin. The Kanawha River Basin drains a large area in southern West Virginia and has its headwaters in North Carolina and Virginia.

The Kanawha River flows in a north to north-northeast direction in the vicinity of the site, and forms the site's western and northwestern boundary (see Figure 1). The Kanawha River is used for barge transportation, and river levels are controlled by a series of dams and locks. The normal pool elevation in the vicinity of the site is approximately 566 feet above sea level which is maintained by the Winfield Dam and Locks located approximately 10.9 miles downstream of the site. Based on published reports, the average volume of flow in the Kanawha River at Charleston is approximately 14,000 cubic feet per second or approximately 9 billion gallons per day.

Major tributaries of the Kanawha River in the area include Elk River, which enters at Charleston about 15 miles upstream of the site, and the Pocatalico River, which enters approximately 3 miles downstream from the site. Armour Creek, a smaller perennial stream, originates at higher elevations and enters the Kanawha Valley upstream of the site. Upon entering the valley, Armour Creek turns sharply to the north paralleling the Kanawha River, and flows several miles before joining the river one mile north (downstream) of the site.

2.4 Site Geology

The site is situated on top of the alluvial terrace, and its topography is relatively flat with total relief of less than 10 feet, except along the riverbank. The riverbank is a steep slope which has a drop in elevation of between 20 and 30 feet along the riverfront. The highest elevations on the site are at the following man-made features: along the riverbank; atop the low flood control levee which parallels the river in the Process Area; and at the closed impoundments in the Waste Treatment Area.

Geologic cross-sections through the site, prepared by Roux & Associates (1995), are provided as Figures A-2 through A-5 in Appendix A. The cross sections have been constructed based on logs from boreholes drilled at the site. The Site Plan Figure A-1 in Appendix A provides the locations of the cross sections. As shown in the cross sections, the alluvial deposits extend to a depth of approximately 40 to 50 feet. Fill material is found to depths ranging from 2 to 25 feet in many parts of the site. The underlying deposits contain beds of silt and clay, silty sand, and sand. The grain size of the deposits coarsens downward with silt and clay found mostly at the top of the deposits, and medium to coarse sand with gravel.

Bedrock encountered directly beneath the site is described in drilling logs as gray siltstone. Weathered bedrock encountered in boreholes is described as weathered shale or clay.

2.5 Site Hydrogeology

The alluvial deposits of the Kanawha River Valley contain the uppermost aquifer at the site. The aquifer is unconfined, and the depth to groundwater generally varies from 15 to 20 feet below ground surface across the facility. Although considerable variability occurs in sediment type in the alluvial deposits, the groundwater within the alluvial deposits is considered to be interconnected and can be characterized as a single aquifer. The "A" wells and "B" wells are considered to monitor the upper and lower part, respectively, of the same aquifer.

Groundwater in the alluvial deposits beneath the facility flows toward the Kanawha River across the entire site. Groundwater contours interpolated from groundwater elevation measurements in the alluvial aquifer are shown in Figure 2. These contours represent similar groundwater elevation contours in both Zone A and Zone B.

Aquifer testing conducted at the site indicates a considerable range in hydraulic conductivity both laterally and vertically in the alluvial deposits. In general, hydraulic conductivity increases with depth in the alluvial deposits. Most hydraulic conductivities measured in Zone A wells in the upper part of the aquifer range from 0.1 to 1 ft/day with values as low as 0.01 ft/day and as high as 24 ft/day. The geometric mean for Zone A wells is 0.51 ft/day. Most hydraulic conductivities measured in Zone B wells in the lower part of the aquifer range from 5 to 10 ft/day with values as low as 2.8 ft/day and as high as 12 ft/day. The geometric mean in Zone B wells is 6.7 ft/day.

There are no known potable supply wells in the vicinity of the site which draw water from the alluvial or bedrock aquifers. Water supplies in the region are derived from surface waters; however, there are no potable intakes along the Kanawha River downstream of the site.

Potable water for the Nitro plant is purchased from the West Virginia American Water Company, whose raw makeup water intake is located on the Elk River (a tributary of the Kanawha River) upstream of the site at Charleston.

2.6 Current Surface Features

The Solutia Nitro site encompasses approximately 122 acres and is divided into two separate areas by Interstate 64: 1) a southern area encompassing approximately 76 acres, which contains the former process area (PA) and past disposal area (PDA) and; 2) a northern area, encompassing approximately 46 acres, that contains the former Wastewater Treatment Area (WTA) which included the wastewater treatment plant and wastewater impoundments (see Figure 3). The northern section formerly encompassed the wastewater treatment system.

The Process Area at the site is largely covered by concrete slabs and asphalt, and surface-water runoff is directed via sheet and shallow concentrated flow to a large collection ditch located to the north before being discharged to the Kanawha River through a permitted outfall culvert. The low levee along the riverbank prevents any overland low from reaching the Kanawha River. The facility sewer and storm water collection system was plugged during the demolition of the facility, which was completed in 2005.

Only a small portion of the Waste Treatment Area is covered with asphalt, and most of that area is covered with vegetation consisting of grass and shrubby growth. No ditches or subsurface drains are present in this area, and much precipitation directly infiltrates into the soil. Storm water runoff from localized areas along the southern boundary of the WTA, along I-64 is collected in a shallow surface conveyance ditch which discharges to the Kanawha River through a permitted outfall.

2.7 Constituents of Concern, Primary Source Areas and Planned Interim Measures

The chief constituents of concern (COCs) at the site are TCDD and trichloroethylene (TCE). The primary source areas of these COCs are the Process Area (PA) and Past Disposal Area (PDA), both located south of I-64, and the Waste Treatment Area (WTA) located north of I-64.

Previous Interim Measures (IMs) performed in the Former 2,4,5-T Manufacturing Area (installation of gravel, asphalt and concrete covers) and the PDA (soil and gravel cover) have improved conditions such that it is currently protective of site users. The A3 Basin is currently covered by a 40 mil. HDPE synthetic rain cover and an additional soil cover of approximately 2 feet to control leaching through stabilized sludges in the basin. The planned additional interim measures are:

- installation of a low-permeability cap and barrier wall around the PDA to physically contain impacted soils and wastes and prevent migration of TCDD in groundwater from the PDA to the adjacent Kanawha River;
- installation of a low-permeability cap and barrier wall around the former process area (the source of TCE);

- installation of a low-permeability cap and barrier wall around an area encompassing the City of Nitro dump and former waste pond.

The locations of the planned additional interim measures barrier walls are shown in Figure 3.

3.0 Pumping Test Data Analysis

Between September 15 and 20, 2010, Potesta and Associates conducted pumping tests in bedrock wells at each of the three source areas at the site. The locations of the three pumping tests are shown in Figure 4. At each location, the drawdown in the bedrock pumping well, an observation well installed in the bedrock, and a Zone B monitoring well were measured with pressure transducers. The drawdown plots at each of the three locations are shown in Figures 5 through 7.

3.1 Pumping Test Data Analysis Procedure

The pressure transducers at each extraction well location were placed near the bottom of the wells. As seen in the three time vs. drawdown plots (Figures 5 through 7), the water level rapidly decreased to an elevation near the well bottom elevation, at which point the head in the extraction wells became constant or varied above a minimum value. The extraction well heads indicate that the pumping rate was limited by the amount of water that could flow into the well, in effect converting the constant-rate pumping tests into either a constant-drawdown pumping test (for the PA) or a variable-drawdown pumping test (for the WTA and PDA). Because there are no commonly available analytical techniques for analyzing a test with such a change, the observation well drawdown responses in each pumping test were simulated in MODFLOW using idealized geometry of the aquifer system as part of this modeling study.

The idealized system for each well consisted of three layers corresponding to each of the three water-producing zones at the site (Zone A, Zone B, and the bedrock). The model layers representing each zone corresponded to the layers used in the groundwater flow model (see Section 3.1.2) except that the bedrock zone thickness was increased to 50 feet during the analysis to increase model stability. A very small, 1-ft horizontal grid was used in the area of the pumping and extraction wells to obtain more accurate drawdown values for the analysis. The horizontal grid size was increased outward from the well locations to no-flow boundaries to the north, east and south. The western boundary was simulated as a constant-head boundary representing the Kanawha River. Drawdowns at all four of the lateral boundaries were negligible at the end of the simulations, indicating that these boundaries had no significant effect on the drawdowns observed at the observation wells.

To simulate the pumping test, the extraction well was represented not as a well, but as a transient head boundary condition. The transient head was set equal to the head measured in the pumping well by the transducer. The fluctuations in the transient head boundary therefore duplicated the measured drawdown history in the pumping well and eliminated the need to know the actual well pumping rate.

The horizontally isotropic hydraulic conductivity (Kx and Ky) of Zone A and Zone B were set at values estimated from slug tests and were not changed (see Table 1). These values were 0.5 ft/d in Zone A and 7 ft/d for Zone B. The vertical hydraulic conductivities (Kz) for these two zones, and both the horizontal and vertical hydraulic conductivity of the bedrock, were varied until the observed drawdowns in the bedrock and Zone B observation wells were matched reasonably well.

The pumping tests were simulated sequentially. First, the measured early drawdown data at the bedrock observation well was matched to simulated drawdown at times before the head in the pumping well reached its minimum value. At early times in a pumping test, drawdowns generally match a confined aquifer drawdown (Theis) curve. The horizontal hydraulic conductivity and specific storage of the bedrock were varied during this period until the simulated drawdown curve matched the measured bedrock observation well drawdown curves, providing an initial estimate of horizontal hydraulic conductivity and specific storage for the bedrock.

In the second step, the vertical hydraulic conductivity in the bedrock was determined. The model bedrock vertical hydraulic conductivity was varied until the simulated drawdown in the bedrock observation well matched the measured drawdowns at times after the initial, early-time period.

Finally, the vertical hydraulic conductivity in Zone A and Zone B were varied in the model until the simulated drawdown at the Zone B observation well matched the measured drawdown in the Zone B observation well.

3.2 Pumping Test Data Analysis Results

The results of the pumping test analysis at the PA and WTA areas are summarized in Table 1. Simulated drawdown curves are plotted against measured drawdowns in Figures 8 and 9. The bedrock horizontal hydraulic conductivities for the wells were fairly close at these two locations (0.51 ft/d at the PA and 0.86 ft/d at the WTA). The bedrock vertical hydraulic conductivities were more different, ranging from 0.0012 ft/d at the PDA to 0.043 ft/d at the WTA, a difference of over 30 times.

The vertical hydraulic conductivity of Zone B was determined to be approximately 0.35 ft/d from the pumping test simulations at the PA. This value represents a vertical anisotropy factor (Kx/Kz) of 20 in Zone B. In the upper elevations of Zone B, where there is more silt and clay, an anisotropy value of 20 is reasonable and possibly even low for a formation of this nature. In the lower elevations of Zone B, where gravel and coarse sands predominate, an anisotropy factor of 20 is probably higher than the true anisotropy of these coarse-grained media.

In Zone A, a vertical hydraulic conductivity of 0.095 ft/d was determined from the pumping test simulation at the PA. This value represents an anisotropy factor of 5 in Zone A, which is probably low for a formation dominated by fine-grained soils.

The drawdown measured in the Zone B well at the PDA was much smaller than the drawdowns measured in the other two wells, resulting in an initial estimate of horizontal hydraulic conductivity at this location that was unrealistically high for a siltstone. The most likely causes of the significantly different Zone B well drawdowns are either a very high horizontal hydraulic conductivity in Zone B or short-circuiting of water from Zone B to the bedrock at a location not close to the Zone B well. Because it is unlikely that the PDA pumping accurately represents hydraulic characteristics of the bedrock, the PDA data was not analyzed further and was not used to establish characteristics of the bedrock formation.

3.3 Uncertainty in Pumping Test Data Analysis Results

The horizontal hydraulic conductivity and specific storage of the bedrock estimated from the pumping test simulations should be fairly accurate, since they were determined with an early-time dataset in which the effects of leakage and storage are small. The vertical hydraulic conductivities determined for the bedrock, Zone A and Zone B are much more uncertain because there are many different sets of vertical hydraulic conductivity values that could result in reasonable matches to measured data. In addition, the following sources of error apply to all of the pumping test data analysis results:

- Although a small model cell size of 1 ft was used in the area of the pumping and observation wells, the actual extraction well radius was smaller than 1 ft. Therefore, some error was introduced by simulating the well with a variable-head cell. Because the model cell size was larger than the actual well diameter, the drawdown induced by the pumping well was applied further from the actual well. This effect leads to greater drawdown in the observation wells than would actually occur, resulting in a minor underestimation of the true hydraulic conductivity.
- Some of the drawdown observed in the pumping well was a result of friction losses through the wellscreen. This greater drawdown resulting from friction losses was applied in the variable-head model cell as if it were caused completely by the formation. The inclusion of the unknown head loss in the variable-head boundary also results in a greater drawdown at the observation wells, causing an underestimation of formation hydraulic conductivity that is more significant than the error caused by the model cell size.
- The horizontal hydraulic conductivity of Zones A and B were fixed at the geometric means of the conductivity values determined from previous hydraulic testing. Because these values varied considerably across the site, the use of the same, fixed values at both of the pumping test locations may have resulted in conductivity values that are different from values that would have been determined if these conductivities had been allowed to vary.
- Only three layers were used in the pumping test simulations because there are three layers in the groundwater flow model. The use of the same number of layers allows the pumping test simulation results to be assigned directly to the flow model layers. However, the use of only three layers in the pumping test simulations reduces the vertical resolution of the pumping test model, and makes the vertical hydraulic conductivities subject to more uncertainty.

Considering the uncertainties in the pumping test data analysis, the pumping test results were used as starting values and general reference values in the calibration of the groundwater flow model. Flow model hydraulic conductivities were allowed to vary from these values, within reason, if other values in the same general range of these values resulted in a better model calibration.

4.0 Model Construction and Calibration

4.1 Conceptual Model

The model domain was conceptualized as a geologically-bounded alluvial valley. Groundwater in the valley originates from recharge in the mountains outside of the valley and from direct recharge to the aquifer within the valley confines. The recharge from the mountains outside the valley flows beneath the valley through the bedrock formation and discharges to the Kanawha River and the lower sections of the alluvium. Direct recharge from precipitation and groundwater in the alluvium discharges either to the Kanawha River to the west or Armour Creek to the east. The flow toward the River and Armour Creek creates a groundwater divide between the river and the creek.

The Kanawha River is maintained at a uniform elevation. Because of the constant river water elevation and because no groundwater has been pumped from the water-bearing units near the site for many years, steady-state conditions were assumed for all simulations. Other elements of the conceptual model are described below.

4.1.1 Lateral Extent of Model Domain

The model domain is the lateral and vertical limit of simulated area. To ensure that the model boundaries do not affect simulation results, the model domain was extended to natural geologic features that can be easily simulated with flow boundary conditions. The extent of the model domain is shown in Figure 10.

The Nitro site lies in an alluvial valley formed by the Kanawha River. The valley is bounded by steep slopes through which streams have excised channels through the mountains. The mountains and the small valleys created by the streams are natural lateral boundaries that form the east and west boundaries of the model domain.

Approximately 2 miles north of the I-64 bridge, the east bank of the Kanawha River approaches the mountains, effectively cutting off the part of the valley in which the site is located. This point is taken as the northern extent of the model domain. The southern edge of the model domain was extended to a similarly narrow point on the eastern alluvial plain about 2 miles south of the I-64 bridge.

4.1.2 Model Layers

The alluvial aquifer and the top of the bedrock are simulated with three model layers. Layer 1 represents Zone A of the alluvial aquifer, Layer 2 represents Zone B, and Layer 3 represents the top of the bedrock. Zones A and B were simulated as unconfined formations while the bedrock, because of expected high anisotropy, was simulated as a confined layer. A conceptual representation of the model layers is provided in Figure 11.

An analysis of soil borings indicates that bedrock is present at an average elevation of approximately 535 feet mean sea level (msl). Although it would be permissible to allow the bedrock to form the lower model boundary, head measurements indicate an upward gradient from the bedrock to Zone B. Therefore, to more accurately simulate groundwater flow patterns near the bottom of Zone B and beneath the Kanawha River, an upper portion of the bedrock was explicitly included in the model domain. The top of the model domain was defined by the digital elevation model (DEM) for the site area.

Throughout most of the domain, Layer 1 extends from the ground surface or the Kanawha River surface to a bottom elevation of 550 ft msl, an elevation that roughly divides the wells screened in Zone A from wells screened in Zone B. Near the mountains to the east and west of the site, Layer 1 gradually diminishes in thickness down to a minimum value of 1 foot at an elevation of 620 ft msl. This elevation is assumed to represent the upper-most limit at which sedimentary material comprising Zone A exists, and forms a limit to the top of Layer 1 in the model (see Figure 11).

The bottom of Layer 1 coincides with the top of Layer 2. The bottom of Layer 2 is set at the top of the bedrock elevation of 535 ft msl throughout most of the model domain. Near the eastern and western extent of the model domain, the thickness of Layer 2 diminishes to a minimum of 1 foot, so that the maximum elevation of Layer 2 in the model is set to 618 ft msl.

The top of Layer 3 is set as the bottom of Layer 2. Boring logs along the barrier wall routes surrounding the individual source areas indicate that the overall average elevation of the shale/siltstone bedrock is approximately 534 ft msl. The data indicate that, although the bedrock elevation is relatively uniform, there are some "holes" and "hills" in the bedrock elevation at some locations. The bedrock elevations from all of the data in the site area are shown in Figure 12.

The bedrock elevation north of I-64 appears to be, on average, a few feet lower than the bedrock elevation south of I-64. North of I-64, the average bedrock elevation is approximately 531 ft msl, while south of I-64, the average bedrock elevation is 535 ft msl. Although the dataset defining the top of the bedrock is sparse on the scale of the model, bedrock elevations in the model area were interpolated to match the point elevations of Zone B/bedrock contacts. The true bedrock elevation is undoubtedly much more variable than the interpolated values used in the model, so that these interpolated values should be considered as a single representation of the greater, but unknown, bedrock elevation variability.

The bottom elevation of Layer 3 was set at an arbitrary elevation of 485 ft msl, creating a 50-ft thick bedrock layer throughout most of the area. The 50-ft thickness allows continuous head contours to be drawn from the bottom of Layer 2 so that groundwater flow paths can be ascertained in cross-section.

4.1.3 Physical Boundaries

Bathymetric data for the Kanawha River indicates that the river cross-section through the alluvium approximates the shape of a rectangle along much of the site river bank (Golder Associates, 2004). Therefore, the sloping bottom of the river near the bank was not explicitly simulated. Instead, river conductance was adjusted to account for the sloping river bottom during model calibration. The bottom of the river was fixed at the average elevation of 535 ft msl, which corresponds to the bottom of Layer 2 and the top of the bedrock. The lateral boundaries of the river were defined by the 570 ft msl elevation contour of the DEM and the river boundaries observed in aerial photographs. The 570 ft msl elevation contour was used instead of the 566 ft msl contour (which corresponds to the normal Kanawha River pool elevation) to account for any slight errors in the DEM that could cause parts of the river to be incorrectly excluded from the river boundary.

The DEM indicates that the ground surface elevations in the vicinity of Armour Creek on the east of the site and just within the eastern model domain are quite low, indicating that this creek probably has a significant effect on groundwater flow. Potesta (2010) also recognized that Armour Creek affects the groundwater potentiometric surface. The upper reaches of Armour Creek, from a point approximately 500 ft north of the I-64 bridge to all points south were treated as a drain, meaning that groundwater can enter Armour Creek but cannot flow from Armour Creek back into the aquifer.

The depth of the widened area of Armour Creek from about 500 ft north of the I-64 bridge to the Kanawha River is not known. This section was therefore treated as a lake, which behaves hydraulically similar to a river but without a limit on bottom elevation. The effect of this section of Armour Creek on groundwater was adjusted during calibration by changing the lake's conductance.

Interpreted heads depicted on the cross-sections prepared by Roux and Associates (1999, see Appendix A) suggest upward flow from the underlying bedrock as a result of an upward hydraulic gradient between the bedrock and Zone B. Some degree of groundwater underflow in the bedrock was used to create these observed conditions. Groundwater underflow from the bedrock was simulated with a general head boundary in Layer 3. The parameters for the general head boundary were adjusted during model calibration.

Precipitation in the Nitro area averages approximately 44 inches/year. A starting value for recharge of 10% of precipitation (4.4 in/yr or 0.01 ft/d) was initially used in the model. The recharge value was adjusted during model calibration. Recharge was set to 0 within the building slabs, roads, paved parking areas, and other locations where recharge was expected to be minimal based on land use determined from aerial photographs. The interim measures call for the installation of geocomposite synthetic covers on all source areas within the boundaries of the barrier walls. Since synthetic capping systems will be installed, these areas were also assigned a recharge of zero. A lower recharge rate within the site was determined through calibration. Evapotranspiration was not explicitly simulated, but is implicitly included in the recharge rate, which represents average net recharge (recharge – evapotranspiration). The final recharge rates used for the three recharge zones in the site area are shown in Figure 13.

4.2 Model Software

The software program MODFLOW 2000 (McDonald and Harbaugh, 1988) was used for the groundwater flow simulations. MODPATH Version 3 (Pollock, 1989) was used for particle tracking. These two programs were run through the graphical user interface Groundwater Vistas, Version 5.41, Build 3 created by Environmental Model Systems Inc.

4.3 Model Domain Horizontal Discretization

The model horizontal grid spacing is shown in Figure 10. To reduce the potential numerical dispersion in case the model is ever used for mass transport simulations, a fairly small horizontal grid spacing of 100 feet was used as a base grid size. A smaller grid size of 25 feet was used within the site to limit potential future numerical dispersion, facilitate more accurate specification of barrier wall boundaries, and provide better resolution of flow patterns in the area of interest. The grid spacing can be expanded outside of the area of interest if needed in the future, since boundaries outside of the area of interest need not be specified with high resolution. However, since typical simulations executed in less than 1 minute, further grid adjustment was not necessary for this project.

4.4 Model Domain Vertical Discretization and Layer Properties

A representative conceptual cross-section of the model grid that illustrates the vertical model domain discretization is provided in Figure 14. Because only steady-state conditions are simulated, the only aquifer property that requires specification was hydraulic conductivity. With regard to hydraulic conductivity, all three model layers were assumed to be homogeneous, horizontally isotropic, and vertically anisotropic. Boring logs from monitoring wells installed at the site indicate that the subsurface is highly heterogeneous in Zone A, especially at shallow elevations. However, site data is not of sufficient density and of sufficient coverage to justify spatial variation of hydraulic conductivity. Therefore, layer properties were assumed to represent spatially-averaged properties.

For Layers 1 and 2, the geometric mean of the slug test data was fixed as the uniform horizontal hydraulic conductivities. The Layer 1 and 2 vertical hydraulic conductivities, and the horizontal and vertical hydraulic conductivities of the bedrock, were set at values determined from the pumping test data analysis. The Layer 3 hydraulic conductivities and vertical hydraulic conductivities in Layers 1 and 2 were adjusted during model calibration. The final values of hydraulic conductivity for all three layers are as follows:

Layer	Kx, Ky (ft/d)	Kz (ft/d)
1 (Zone A)	0.5	0.095
2 (Zone B)	7.0	0.35
3 (Bedrock)	0.25	0.015

The locations of boundary conditions in Layers 1 and 2 are shown in Figures 15 and 16, respectively.

4.5 Model Calibration

The groundwater flow model was calibrated by adjusting boundary condition parameters until the observed heads matched the simulated heads to a reasonable degree of accuracy. The simulated potentiometric surface contours were also compared to the interpolated potentiometric surface contours based on site measurements as a qualitative check on calibration. The July 2003 set of measured heads was used for calibration because this dataset is comprehensive and is conducive to a reasonably regular interpolated potentiometric surface similar to a simulated potentiometric surface. Parameters were adjusted to match hydraulic heads measured in all three groundwater zones.

4.5.1 Calibration Accuracy Measures

The mean error (ME), mean absolute error (MAE), and the normalized root mean squared error (RMSE) were used as quantitative measures of how well the simulated heads matched the observed heads. The definitions of these errors are as follows (Anderson and Woessner, 2002):

$$ME = \frac{1}{n}\sum_{i=1}^{n}(h_m - h_s)_i$$

$$MAE = \frac{1}{n}\sum_{i=1}^{n}\left|(h_m - h_s)\right|_i$$

$$RMSE = \frac{\sqrt{\frac{1}{n}\sum_{i=1}^{n}(h_m - h_s)_i^2}}{(h_{max} - h_{min})}$$

where n is the number of measurements, h_m is the measured head, h_s is the simulated head, h_{max} is the maximum observed head, and h_{min} is the minimum observed head.

The ME is the average difference between measured and simulated heads. A well-calibrated model should result in a ME near zero, indicating that the model heads are not biased high or low so that all of the errors cancel out. If the ME is negative, the average elevation of the simulated potentiometric surface is higher than the measured potentiometric surface, or is biased high. A positive value of ME indicates that the potentiometric surface is biased low.

The MAE is the average of the absolute values of the errors and is always positive. It is a measure of the average deviation between observed and simulated values. The smaller the MAE, the better the simulated heads match the measured heads.

The RMSE is the standard deviation of the differences between measured and simulated heads normalized by the range of the observed heads. Generally, a well-calibrated model should have an RMSE value less than 10%.

All of these quantitative error measurements report the average error in the fit of simulated to measured heads, but no information about the distribution of the error. Spatial trends in errors can indicate a poorly calibrated model even if the three calibration statistics are reasonable. To understand the distribution of errors, three plots were also examined to ensure that there were no error trends that could indicate a poor model calibration.

The first type of plot is simply a plot of the residuals (difference between measured and simulated heads) throughout the domain at each observation well location. A good model calibration should result in a random distribution of residuals around zero, with no evident spatial trend in the residual sign or magnitude. Plots of residuals in the model domain are shown on Figures 17, 18, and 19 for Zone A, Zone B, and the bedrock, respectively.

The second type of plot used is a scatter plot, which is a plot of the simulated heads against the measured heads. A perfect calibration would result in all of the points lying on a straight line at a 45 degree angle to the x-axis. A reasonable calibration should result in random scatter of the data around the 45 degree line. If the best fit line of the measured and observed heads does not have a 1 to 1 slope, the calibration is biased to either low-head or high-head data. If more data points appear above or below the 45 degree line, then simulation is either over-predicting or under-predicting the average head. A scatter plot for this model calibration is shown in the top of Figure 20.

The third plot is a plot of residuals on the y-axis vs. observed heads on the x-axis. In this plot, a good calibration will result in a random distribution of residuals above and below zero, with no bias in error toward either high or low observed heads. A residual plot for this model calibration is shown in the bottom of Figure 20.

4.5.2 Calibration Procedure

Before calibration began, wells were assigned to Layer 1, Layer 2 or Layer 3 based on their screened intervals. Table 2 shows the layer assignments of each well for which groundwater elevation was available based on the well screened intervals. With a few exceptions, wells that straddled Layer 1 and Layer 2 were excluded from the dataset since the head in these wells represents a combined head contribution from both Zone A and Zone B. Well WT-15A was also eliminated from the dataset because it is screened in a perched aquifer (Roux Associates, 1999). Finally, several other wells were excluded from the dataset because the heads in these wells did not match the trend of heads observed in other wells in their vicinity. These wells may be screened in more isolated portions of the aquifer or could represent small scale variability in aquifer properties or boundary contributions that cannot be accurately simulated.

Only a few boundary condition parameters are based on site data. Most of the parameter values were determined through model calibration. Because many of these boundary condition parameters are highly correlated, the model calibration is non-unique; i.e., there are many combinations of boundary condition parameters other than those adopted for final project simulations that could result in an equally acceptable model calibration. Because there are no flux stresses applied to the groundwater system, flux data that could reduce the number of possible calibration parameter sets is also unavailable. This uncertainty in model calibration is typical of groundwater flow models and should always be kept in mind when evaluating model predictions.

Model parameters that are based on site measurements and that were not adjusted during calibration were:

- Hydraulic conductivity of Zones 1 and 2 (Layers 1 and 2 of the model). Values were fixed at the geometric mean of slug test results.
- River bottom elevation (fixed at 535 ft msl based on an average of bathymetric data for the site area);
- River stage (fixed at 566 ft msl based on site measurements).

Table 3 lists the initial values of all other model parameters, along with the basis for these values. After an initial run using the starting set of parameters, calibration was achieved by adjusting river conductance, general head boundary elevation and hydraulic conductivity, lake hydraulic conductivity, drain hydraulic conductivity, and recharge until the shape of the simulated potentiometric surface was a reasonable match to the shape of the interpolated July 2003 potentiometric surface. These parameters were then further adjusted to match measured heads to simulated heads in observation wells, minimize the quantitative error measurements, and eliminate any spatial trends in the distribution of residuals in each zone.

4.5.3 Evaluation of Final Model Calibration

Final simulated heads and measured heads are listed in Table 3, along with the calibration error statistics. The overall ME of -0.01 indicates that the average of the simulated heads is slightly higher than the average of the measured heads, but is very close to zero. The overall MAE of 0.47 indicates that the average difference between measured and simulated heads is about ½ foot. The RSME of 8.3% is less than the calibration goal of 10%. These statistics indicate that overall, the model calibration results in heads that are reasonably close to the measured heads.

Figures 17 through 19 show the distribution of the residuals in Layers 1, 2, and 3. The distribution of errors in Layer 1 is fairly even, indicating a lack of bias in predicted heads. The ME calculated for Layer 1 in Table 2 is close to zero, indicating a lack of bias in simulated heads either high or low. The RSME of 8.2% indicates a well-calibrated model for Layer 1.

The ME of -0.1 in Layer 2 indicates that the simulated heads are, on average, low by about a tenth of a foot. The RSME of 10.4% is slightly above the calibration goal of 10%, suggesting some improvement in Layer 2 calibration might be achievable, possibly by adjusting river conductance or refining the model grid near the river. However, because of the close qualitative match between measured and simulated heads in Layer 2, additional effort to achieve a better match to observed heads is unlikely to significantly change groundwater flow paths, so that the additional effort is probably not warranted.

Three of the four simulated heads in Layer 3 are higher than the measured heads, suggesting that some bias towards high heads exists in Layer 3. However, the bias is relatively low and the RSME of 8.8% indicates an acceptable calibration.

Figure 20 shows both a plot of observed vs. simulated heads (upper plot) and a plot of residuals vs. observed heads (lower plot). The simulated vs. observed head plot confirms that there is a slight bias towards low simulated heads, with most of the error occurring in the head range of 569 to 572 ft msl. Most of the observed heads in this range occur close to the Kanawha River.

Similarly, the observed head vs. residual plot indicates a slight bias in the same range of observed heads.

The simulated potentiometric surface for Layer 1 is overlaid on the potentiometric surface interpreted by Potesta (2010) in Figure 21.

Although these results indicate that some improvement in model calibration might be achieved, there are two factors limiting further significant improvements in calibration. The first factor is the measured vertical flow gradients that indicate changing heads with depth. Because Zones A and B are both simulated as a single layer in which the head is constant with depth within the layers, some depth-related error in the calibration is unavoidable.

The second factor that may be causing increased errors along the Kanawha River is the fact that Zones 1 and 2 are both unconfined aquifers discharging to a surface water body. Unconfined aquifer discharge forms a seepage face along the discharge surface. Because MODFLOW uses a hydraulic approach to simulating unconfined aquifer (the Dupuit assumptions for unconfined flow), the seepage face is not accurately represented in the model, although discharge rates are. A seepage face would result in actual heads being higher than simulated heads, which is the case in most of the wells near the river.

4.6 Final Model Parameters and Simulated Potentiometric Surface

Final parameters for both the layers and all boundary conditions are provided in Table 3. The table also indicates which parameters were changed during calibration, and what the initial values were assumed to be before they were altered. The table also provides the basis for the value of all parameters in the model.

4.7 Model Limitations

The quality of groundwater flow model calibration is about average for a typical groundwater flow model. The predicted groundwater potentiometric surface closely matches the observed potentiometric surface in most areas of the model domain, and the matches between the simulated and observed hydraulic heads are reasonable. The groundwater flow paths predicted by the model appear intuitively correct and match the behavior typically observed for the boundary conditions used. The model runs quickly and appears to be numerically robust, making it a useful exploratory tool. These characteristics indicate that the model appears to be a good representation of the groundwater flow system at the site, and should be a reliable tool for estimating flow paths and the effects of barrier wall construction around the source areas.

As with most groundwater flow models, the groundwater flow model created for the Nitro site exhibits some degree of non-uniqueness; i.e., there are other parameter sets that result in an equally good model calibration. This non-uniqueness means that there is some uncertainly regarding the potentiometric surface and groundwater flow paths. In addition to the issue of non-uniqueness, some limitations of the model that should be considered when interpreting the results are:

- The model represents the heterogeneous, anisotropic subsurface as a layered, homogeneous and isotropic system. Therefore, the model results represent spatially averaged groundwater flow conditions. Small-scale variability in groundwater flow paths caused by local heterogeneities and anisotropy are not represented in the model.
- The seepage face at the Kanawha River is not well-represented by MODFLOW, which cannot correctly simulate a seepage face. Therefore, caution should be used when interpreting heads at the Kanawha River boundary, although groundwater fluxes into the river should be accurate.
- The final calibrated value for horizontal hydraulic conductivity in the bedrock is at the upper end of the range reported for a typical sandstone. Freeze and Cherry (1979) report typical values of approximately 0.0003 to 3 ft/d for a sandstone, whereas the calibrated value for the bedrock in the model is 0.25 ft/d. Much of the bedrock is described as siltstone, claystone, or shale, which is even less permeable than sandstone. While the calibrated value is based on adjustments of conductivities measured in pumping tests at the site, it is possible that short-circuiting of groundwater in the pumping tests lead to an overestimate of the true bedrock hydraulic conductivity. The high value of hydraulic conductivity means that the model is very conservative with regard to flow in the bedrock (i.e., the model may over-estimate flow in the bedrock). The effects of this conservatism are discussed in the next section.

5.0 Project Simulations

The calibrated groundwater flow model was used to answer the following questions:

- What effect will the proposed barrier walls have on groundwater flow patterns and elevations under the selected wall configurations, and specifically, could wall construction cause flooding upgradient of the source areas?
- What effect will altered groundwater flow patterns have on the existing monitoring well network, and specifically:
  - Could altered groundwater flow patterns following barrier wall construction adversely affect groundwater that is not currently impacted?
  - What are the optimum locations for monitoring well pairs to evaluate the performance of the Interim Measure by monitoring the hydraulic head difference across the barrier walls?
- What pumping rate within each barrier wall is needed to maintain an inward hydraulic gradient?
- Would changes in the Kanawha River stage have a significant impact on groundwater flow patterns?

These questions are addressed in the following sections.

5.1 Effect of Barrier Walls on Groundwater Flow Patterns and Elevations

For purposes of the groundwater flow simulations, barrier wall hydraulic conductivity was set to 3.0×10^{-8} cm/s based on vendor estimates of what can be achieved at the site. The barrier walls were simulated using the horizontal flow barrier boundary condition that is part of MODFLOW. The horizontal flow barrier package allows the wall to be placed on any number of sides of each grid cell to form a continuous barrier that isolates grid cells within.

Simulations indicate that installation of the four barrier walls will affect both groundwater flow patterns and hydraulic heads in the site wells. The effect of the barrier walls on the potentiometric surface and groundwater flow patterns in Zone A can be seen by comparing Figures 22 and 23. The barrier walls act as partial dams that channel water through the aquifer between the source areas. As a result, groundwater heads increase throughout the area upgradient of the source areas. Similar effects can be seen in the Zone B flow patterns illustrated in Figures 24 and 25.

It is important to note that all of the groundwater pathlines shown in figures represent two-dimensional projects of three-dimensional groundwater flow. Because the model has three layers with spatially varying groundwater recharge and boundary conditions that do not extend into all layers, vertical flow in some parts of the model domain is significant. As a result, particles that originate in one layer may follow flow paths that reside in a different layer for most of their flow history. This fact should be kept in mind when interpreting the flow paths shown in the figures.

The effect of barrier wall construction on hydraulic heads throughout the area is small, indicating that the barrier walls should not result in flooding of any on-site or off-site areas. The magnitude of the predicted water level change in individual wells is shown in Table 4, and contours of water level changes across the site are shown in Figures 26 and 27. The greatest changes in head occur in wells that will be within the barrier walls. Of the wells outside of the barrier walls, the greatest calculated head change is in wells GW-6A and GW-6B at slightly more than 1 foot. Generally, groundwater elevations are predicted to change by less than 1 foot across the entire area following wall construction, with most areas experiencing less than 0.5 feet of groundwater elevation change.

5.2 Effect of Barrier Walls on the Groundwater Monitoring Network

The effect of barrier wall construction on the monitoring network can be seen by examining the location of the current monitoring wells in relation to the barrier walls and groundwater flow paths in each layer. The location of monitoring wells and flow paths in Zone A before barrier wall construction are shown on Figure 28. Seven of the current monitoring wells (GW-1A, GW-5A, GW-9A, GW-10A, GW-11A, GW-12A, and GW-7A) will be located within the barrier walls after interim measures construction and would need to be replaced if they are needed for monitoring of groundwater quality outside of the barrier walls. Five of these wells could be utilized for pumping inside the barrier walls to maintain an inward hydraulic gradient (see Figure 30). The wells inside the walls could also be used to monitor groundwater levels or groundwater quality within the barrier walls.

The groundwater flow paths shown in Figure 23 indicate that the wells in the southern part of the site southwest of the process area will most likely be affected by the installation of the barrier walls in Zone A. Based on the results of the model, the effect would be minor. The simulated pathlines in Zone A indicate that the installation of the barrier wall around the PA will deflect groundwater flow in a more westerly direction around the southern side of this area. This more westerly flow could result in wells MW-3A, MW-19A and GW-3A being slightly more downgradient of the southern side of the PA, and may result in small changes in COC concentrations in these wells. However, because the groundwater deflection is close to the exterior sides of the barrier wall at the PA, significant changes to concentrations in these wells are unlikely. The monitoring wells along the southern side of the PA (MW-1A, MW-17A, GW-2A, MW-18A, MW-23A, and EW-8) are unlikely to be affected by the PA barrier wall construction.

In Zone B, the effect of the barrier walls on the monitoring well network is most significant between the PA and the PDA (see Figure 25). Concentrations in well GW-4B are likely to change as groundwater flowing past this well originates further north and east, in areas north of the PA.

Although the altered flow patterns may have a very small effect on existing monitoring well concentrations, the concentrations may not be significant enough to warrant installation of new wells at this time based on potential concentration changes.

The installation of the barrier walls will create the need for new monitoring wells to ensure that an inward hydraulic gradient is maintained across the four barrier walls. Because groundwater flow is generally westward across the source areas, groundwater will tend to mound on the exterior of the east wall at each source area, maintaining an inward hydraulic gradient along the east wall and halfway down the north and south walls. On the west side of source areas, the head in the aquifer will be at its lowest, and it is along the west wall that pumping inside the wall is needed to maintain an inward hydraulic gradient.

The gradient along the western wall will be smallest, and will have the greatest potential for reversing from an inward gradient to an outward gradient. Therefore, monitoring the head gradient across the western wall of each area should provide the worst-case (lowest) measurement of the gradient across the barrier walls in each source area. However, if groundwater elevations show anomalous variability, then additional monitoring wells across the north and south walls could also be installed to ensure an inward gradient along the entire wall length.

Potential locations of head gradient monitoring wells are shown in Figures 28 and 29. Pairs of monitoring wells are proposed for the western, downgradient sides of the barrier walls, and the north and south sides (if needed). These wells were located to be as far away from pumping wells on the inside of the barrier walls as possible, where heads within the walls would be expected to be at their highest, resulting in a more conservative demonstration of an inward gradient (i.e., not directly influenced by the cones of depression immediately surrounding the pumping wells).

5.3 Effect of Pumping Within Walls on Mass Flux to River

Groundwater flows into the alluvial aquifer from bedrock underflow through Zone B below and from recharge through Zone A above. Although the low-permeability cap over the source areas should reduce downward infiltration to negligible volumes, the model predicts that significant underflow from the underlying bedrock will create an outward hydraulic gradient from the source areas to the groundwater outside of the barrier walls. If the predicted volume of underflow from the bedrock is accurate, then groundwater would have to be extracted from within the barrier walls to maintain an inward horizontal hydraulic gradient that will prevent impacted groundwater from escaping the source areas.

To determine the minimum pumping rate needed to maintain an inward hydraulic gradient, pumping wells were placed within each source area. All wells were placed in Layer 1. Where possible, existing wells were utilized for pumping. Particles were then placed in each cell within the barrier wall at the bottom and top of Layer 2 and at the water table to ensure that water from neither zone escapes through the barriers. The pumping rates of the wells were then gradually increased from a low rate until no particles escaped through the walls.

Because all of the wells were placed in Layer 1, pumping rates were limited to those that would maintain the water level above the bottom of the wellscreens. To keep the pump submerged and provide a minimum safety factor, pumping rates were restricted to rates that resulted in at least a 5-foot water column above the bottom of the screens in the wells. The bottom of the screen in all new wells was assumed to be 550 ft msl, corresponding to the bottom of Zone A.

The drawdown reported by the model is an average drawdown for the entire model cell. The actual drawdown at a pumping well will be much greater than this average drawdown. The drawdown at the wells was estimated from the model cell drawdown using the Theim equation method described by Anderson and Woessner (2002) for an unconfined aquifer:

$$h_w = \sqrt{h_{avg}^2 - \frac{Q}{\pi K}\ln\left(\frac{r_e}{r_w}\right)}$$

where:

h_w = head at the pumping well screen;
h_{avg} = average head in the model cell;
Q = well pumping rate;
K = hydraulic conductivity;
r_e = effective radius of model cell;
r_w = well radius.

The effective model cell radius (r_e) is calculated from the cell size as:

$$r_e = 0.208a$$

where a is the length of a grid cell side. The number of wells and pumping rates determined for each unit are:

Site Area	Number of Wells	Total Flow Rate (gpm)
WTA	3	0.5
PDA	4	1.2
PA	2	0.7
Total	**9**	**2.4**

Additional details concerning the performance of each simulated well established by this procedure are shown in Table 5. The locations and pumping rates of the wells are depicted graphically in Figure 30. More detailed views of the capture zone of wells within the barrier walls at each area are shown in Figures 31 through 33.

As discussed in Section 4.7, the calibrated value for horizontal hydraulic conductivity of the bedrock is at the upper range of hydraulic conductivity for a sandstone, and greater than the upper range of hydraulic conductivity for a siltstone or shale. If the actual horizontal hydraulic conductivity of the bedrock is significantly lower than the simulated value, then groundwater underflow into the enclosed source areas could be negligible. Under these conditions, the pumping rate within the source areas could be significantly lower than the calculated rates while still maintaining an inward hydraulic gradient. Therefore, the rates reported above are likely conservative estimates of the actual pumping rate needed.

If no groundwater is pumped from within the barrier walls, the model indicates that substantial reductions in groundwater flux through the source areas (and therefore, mass loading to the Kanawha River) will be achieved, even with an outward hydraulic gradient. To determine the effect of the pumping rate from within the barrier walls on mass loading to the Kanawha River, the pumping rates in the wells within the barrier walls were reduced from the values shown in Table 5 to zero in ten equal increments. After each pumping rate reduction, the total reduction in flow through the source areas was calculated. A plot of flow reduction versus pumping rate is shown in Figure 34.

The pumping rate analysis indicates that, in the absence of any pumping within the barrier walls, the groundwater flow through the source areas will be reduced by 99.7%, which means that the mass loading to the Kanawha River from the source areas will be reduced by a similar amount. As the total pumping rate within the walls is increased, the groundwater flow through the source areas decrease as more water within the barrier walls is captured. For example, at a pumping rate of 1.2 gpm, groundwater flow through the source areas is reduced by 99.88%, so that the flow through the source areas is 0.12% of the flow in the absence of the walls. At a total pumping rate of 1.9 gpm, the flow through the source areas will be reduced by 99.96%, so that the flow through the source areas is 0.04% of the flow in the absence of the walls.

Reductions in mass loading to the Kanawha River can also be expressed as a "mass reduction factor," defined as the ratio of the flow rate through the source areas before construction of the barrier walls to the flow rate through the source areas following barrier wall construction. The mass reduction factor (MRF) reflects the factor by which mass entering the river is reduced. Higher MRFs indicate a greater degree of mass reduction.

With no pumping from within the walls, the MRF is approximately 290, indicating that the mass predicted to discharge to the Kanawha River from the source areas will be 290 times less than the current mass discharge. The MRF increases slowly with increasing pumping rate at first, but then increases more rapidly as the Table 5 pumping rates are approached. As shown in Figure 35, a MRF of approximately 2,000 is achieved with a pumping rate of 1.8 gpm, with an ultimate MRF of nearly 16,000 at the Table 5 pumping rate.

5.4 Effect of Kanawha River Stage on Groundwater Flow Patterns

The water level in the Kanawha River is maintained at a constant water level. To determine how potential changes in the Kanawha River water level might affect groundwater flow patterns, simulations were performed with the Kanawha River stage 2 feet higher than the current elevation of 566 ft msl, 5 feet higher than the current elevation, and 2 and 5 feet lower than the current elevation. All of these simulations were steady-state simulations with the barrier walls installed around the three source areas.

Figures 36 through 39 show the effect of the four hypothetical changes in Kanawha River stage on site groundwater flow patterns. The higher river stages have the effect of moving the groundwater divide that separates groundwater flowing into the river from groundwater flowing into Armour Creek further west, toward the Kanawha River. However, the simulations indicate that the groundwater flow patterns adjacent to the source areas are not significantly affected by the higher river stage.

Simulations indicate that lower river stages have minimal effect on groundwater flow patterns. With a lower river stage, the groundwater divide shifts more east toward Armour Creek. As a result, groundwater flows more directly to the Kanawha River.

The effect of changes in river stage on hydraulic heads in individual wells is shown in Table 6. In general, higher river stages result in higher hydraulic heads and vice versa. The head changes are not directly proportional to the river stage, but vary according to distance from the river, nearness of other boundaries, and amount of recharge in the well vicinity.

6.0 Conclusions and Recommendations

In general, the model indicates only small changes in groundwater flow patterns and hydraulic heads across the area following barrier wall construction. The model results appear to be realistic given the complex aquifer geology, limitations in the data, and the large amount of subsurface heterogeneity. Conclusions regarding the specific project objectives are provided in the following sections.

6.1 Effect of Barrier Walls on Hydraulic Heads and Groundwater Flow Patterns

Groundwater flow simulations using the flow model indicate that construction of the interim measures barrier walls will not have a significant impact on hydraulic heads or groundwater flow patterns in either Zone A or Zone B. Outside of the barrier walls, groundwater elevations across the site will increase by less than 1 foot across the site, and in most areas, the increase in heads will be less than 0.5 feet. This small increase in groundwater elevations would not cause flooding on the site or on any adjacent properties.

6.2 Effect of Altered Groundwater Flow Patterns on Monitoring Well Network

The simulations indicate that, following barrier wall construction, groundwater will be deflected around the barrier walls, but the deflection is generally limited to the immediate area surrounding the walls. The altered groundwater flow patterns are unlikely to result in any impacts to groundwater that is not already impacted by past site activities. The altered groundwater flow patterns should not significantly affect the current monitoring well network outside of the immediate area of the barrier walls.

Because groundwater flows in a generally westerly direction, groundwater will mound slightly on the outside of the eastern side of the barrier walls constructed around each source area. The mounding will create a natural inward hydraulic gradient across the east wall and the eastern half of the north and south walls. Along the west barrier wall, the opposite effect will occur, and the groundwater gradient will naturally tend to be outward across the west wall. Pumping groundwater from within the barrier walls will be needed to maintain an inward hydraulic gradient across the west barrier walls.

The simulations indicate that the hydraulic gradient across the west wall will be the smallest gradient at any point along the wall. Therefore, monitoring the head gradient across the west wall should provide a "worst-case" estimate of the gradient across the wall, barring any anomalies in the potentiometric surface. Additional wells on the western half of the north and south walls could also be installed if needed.

6.3 Effect of Pumping Within Walls on Mass Flux to River

Simulations indicate that a total pumping rate of 2.4 gpm is needed to maintain an inward hydraulic gradient at the source areas following installation of barrier walls that exhibit a hydraulic conductivity of 3×10^{-8} cm/s. The number of wells and pumping rates determined for each unit are:

Site Area	Number of Wells	Total Flow Rate (gpm)
WTA	3	0.5
PDA	4	1.2
PA	2	0.7
Total	**9**	**2.4**

The simulations indicate that the inward hydraulic gradient can be maintained with pumping only in Zone A. Four existing Zone A wells can be utilized in the pumping system. These pumping rates are probably conservatively high because the hydraulic conductivity of the bedrock could be significantly lower than the value determined from pumping tests and model calibration.

6.4 Effect of Kanawha River Stage on Groundwater Flow Patterns

A rise in the Kanawha River stage of 2 feet is unlikely to have a significant impact on site groundwater flow patterns. A river stage increase of 5 feet would increase hydraulic heads in wells at the site and move the river/Armour Creek groundwater divide further west. A lowering of the Kanawha River stage would lower hydraulic heads across the site but would not impact groundwater flow patterns significantly.

6.5 Model Limitations

The groundwater flow model appears to be a good representation of the groundwater flow system at the site, and should be a reliable tool for estimating flow paths and the effects of barrier wall construction around the source areas. As with any model, there is some uncertainly regarding the potentiometric surface and groundwater flow paths because the model is non-unique; i.e., there are parameter sets other than the calibrated set used in this model that would also result in a reasonable model calibration. In addition to the issue of non-uniqueness, some limitations of the model that should be considered when interpreting the results are:

- The model represents the heterogeneous, anisotropic subsurface as a layered, homogeneous and isotropic system. Therefore, the model results represent spatially averaged groundwater flow conditions. Small-scale variability in groundwater flow paths caused by local heterogeneities and anisotropy are not represented in the model.
- The seepage face at the Kanawha River is not well-represented by MODFLOW, which cannot correctly simulate a seepage face. Therefore, caution should be used when interpreting heads at the Kanawha River boundary, although groundwater fluxes into the river should be accurate.
- The final calibrated value for horizontal hydraulic conductivity in the bedrock is high for a sandstone/siltstone/shale formation, so that flow in the bedrock could be much smaller than the simulated flows. The main effect of a much lower bedrock hydraulic conductivity would be to reduce the pumping rate necessary to create an inward hydraulic gradient across the barrier walls at each source area.

- There is some uncertainty in the exact location of the groundwater divide between Armour Creek and the Kanawha River. Because the location of the groundwater divide probably changes seasonally and over longer periods with changes in precipitation, this uncertainty should be considered when interpreting groundwater flow patterns near the divide.

7.0 References

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TABLES

Tables

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Table 1
Values of Hydraulic Conductivity and Storage Parameters Estimated from Pumping Test Data

Location	Layer	Hydraulic Conductivity (ft/d) Kx	Ky	Kz	Ss (1/ft)	Sy	Porosity
Waste Treatment Area	1	0.5	0.5	0.05	1.0E-04	0.15	0.4
Waste Treatment Area	2	7.	7.	0.35	2.3E-06	0.3	0.35
Waste Treatment Area	3	0.86	0.86	0.043	4.8E-06	0.009	0.01
Process Area	1	0.5	0.5	0.095	1.0E-04	0.15	0.4
Process Area	2	7.	7.	0.35	2.3E-06	0.3	0.35
Process Area	3	0.51	0.51	0.0012	2.6E-07	0.009	0.01
Geometric Mean		0.66	0.66	0.007			
Arithmetic Mean					2.5E-06		

Key to Shading:

Value based on previous slug tests.

Value estimated using professional judgment.

Value based on results of PA pumping test analysis.

Value determined through analysis of pumping test data at location indicated.

Note: Ss = specific storage; Sy = specific yield.

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Table 2
Well Layer Assignments, Measured Groundwater Elevations, and Calibration Results

Well	X (ft)	Y (ft)	Model Layer	Groundwater Elevation (ft msl)		
				Observed*	Simulated	Residual
MW-23A	1725558	524298	1	570.87	570.87	0.00
MW-3A	1725627	524444	1	570.23	570.58	-0.35
MW-10	1726673	524396	1	573.64	573.17	0.47
MW-2A	1726268	524030	1	573.60	573.21	0.39
MW-13	1727028	523986	1	574.39	574.84	-0.45
MW-11A	1727519	524536	1	574.19	574.51	-0.32
MW-5A	1726093	525335	1	569.45	569.53	-0.08
MW-14	1727176	525414	1	573.37	572.30	1.07
MW-6A	1726407	525751	1	569.80	568.95	0.85
TD-3	1728622	527563	1	566.82	567.28	-0.46
TB-1	1728672	527430	1	567.87	568.20	-0.33
TD-5	1728548	527583	1	566.92	566.86	0.06
MW-22R	1726489	525938	1	567.96	568.25	-0.29
MW-19A	1725679	524615	1	569.07	570.25	-1.18
WT-1	1729528	526816	1	572.07	572.03	0.04
WT-13A	1727984	527257	1	566.72	567.11	-0.39
MW-20A	1725920	525119	1	569.70	569.71	-0.01
MW-21A	1726215	525531	1	567.53	569.22	-1.69
MW-18A	1725987	524125	1	572.29	572.34	-0.05
MW-17A	1726701	523865	1	574.07	574.54	-0.47
WT-10A	1729168	526382	1	572.50	572.87	-0.37
WT-8A	1729803	527781	1	570.40	569.35	1.05
WT-9A	1729299	526983	1	571.84	571.74	0.10
MW-3B	1725628	524450	2	571.14	570.57	0.57
MW-2B	1726273	524029	2	573.40	573.23	0.17
MW-18B	1725982	524128	2	572.37	572.32	0.05
WT-8B	1729804	527777	2	570.43	569.25	1.18
WT-2	1729281	526339	2	572.77	572.85	-0.08
MW-19B	1725681	524620	2	571.50	570.24	1.26
WT-10B	1729164	526384	2	572.41	572.75	-0.34
MW-21B	1726218	525535	2	569.41	569.19	0.22
MW-6B	1726399	525759	2	568.48	568.89	-0.41
WT-4B	1728804	527422	2	569.49	568.53	0.96
WT-7B	1728670	527647	2	566.72	566.83	-0.11
WT-5B	1728999	527770	2	567.18	567.23	-0.05
WT-9B	1729292	526986	2	570.33	571.60	-1.27
WT-10C	1729160	526386	3	572.49	572.65	-0.16
WT-9C	1729285	526990	3	571.17	571.47	-0.30
WT-8C	1729807	527773	3	570.07	569.20	0.87
WT-7C	1728668	527645	3	566.69	567.09	-0.40

Summary of Calibration Statistics			
Dataset	ME	MAE	RSME
All Wells	-0.01	0.47	8.3%
Layer 1 Wells Only	-0.10	0.46	8.2%
Layer 2 Wells Only	0.17	0.51	10.4%
Layer 3 Wells Only	0.00	0.51	8.8%

* Measured heads are from July 2003.
Note: Coordinates are State Plane, West Virginia South, NAD 83, U.S. survey feet.

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Table 3
Initial and Post-Calibration Model Parameters

Parameter	Units	Layer or Zone	Initial Value	Final Value	Basis of Value/Comments
Model Domain Characteristics					
Dimensions	ft	X	NA	15,000	Set to encompass area of interest.
		Y	NA	24,000	Set to encompass area of interest.
Area	mi^2	NA	NA	12.9	Calculated.
Coordinates of lower left corner of grid (State Plane Coordinates, West Virginia South, NAD 83, U.S. survey feet).	ft	Easting	NA	1,721,248	Set to encompass area of interest.
	ft	Northing	NA	513,566	Set to encompass area of interest.
Rows	NA	NA	NA	388	
Columns	NA	NA	NA	307	
Layers	NA	NA	NA	3	
Number of active cells	NA	NA	NA	196,956	
Grid Spacing					
Default horizontal grid spacing	ft	NA	NA	100	Set to relatively small value throughout domain to support potential mass transport simulations.
Refined horizontal grid spacing in site area	ft	NA	NA	25	Set to a smaller value within area of interest to more accurately simulation boundaries and provide better precision on head contours.
Coordinates of lower left corner of refined grid (State Plane Coordinates, West Virginia South, NAD 83, U.S. survey feet).	ft	X	NA	1,725,348	
		Y	NA	523,416	

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Table 3
Initial and Post-Calibration Model Parameters

Parameter	Units	Layer or Zone	Initial Value	Final Value	Basis of Value/Comments
Layer Properties					
Number of layers	NA	NA	NA	3	Layers represent Zone A, Zone B, and bedrock.
Layer elevations	ft msl	1 (top)	NA	566 to 620	Set as digital elevation map elevation with maximum value of 620 ft msl.
	ft msl	1 (bot)	NA	550 to 619	Most cells have bottom elevation at 550 ft msl. Elevation in cells near mountains and along stream valley walls increase with rising elevation.
	ft msl	2 (bot)	NA	527 to 618	Most cells have bottom elevation at 535 ft msl corresponding to the top of bedrock as determined from soil borings. Elevation in cells near mountains and along stream valley walls increase with rising elevation. Cells in the site area vary continuously from 527 to 541 ft MSL based on Kriging of alluvium/bedrock contact.
	ft msl	3 (bot)	NA	485	Set to 50 feet below average bedrock elevation as determined from soil borings.
Hydraulic conductivity (horizontal)	ft/d	1	NA	0.5	Geometric mean of 13 slug tests.
	ft/d	2	NA	7.0	Geometric mean of 8 slug tests.
	ft/d	3	0.51	0.25	PA pumping test data analysis. Adjusted during calibration.
Hydraulic conductivity (vertical)	ft/d	1	0.05	0.1	1/10 of horizontal K. Adjusted during calibration.
	ft/d	2	0.35	0.35	PA pumping test data analysis.
	ft/d	3	0.0012	0.015	PA pumping test data analysis. Adjusted during calibration.

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Table 3
Initial and Post-Calibration Model Parameters

Parameter	Units	Layer or Zone	Initial Value	Final Value	Basis of Value/Comments
Kanawha River Boundary					
Stage	ft msl	1	NA	566	Roux (1999).
		2	NA	566	Roux (1999).
Bottom Elevation	ft msl	1	NA	550	Boring logs.
		2	NA	535	Boring logs.
Sediment Thickness	ft	1	NA	1	Golder Associates (2004)
		2	NA	1	Golder Associates (2004)
Sediment hydraulic conductivity	ft/d	1	1	0.014	Initial value set to high value, then adjusted during calibration. Varies by river segment.
		2 (Adjacent to Site)	1	0.00036	Initial value set to high value, then adjusted during calibration. Varies by river segment.
Recharge (Layer 1 only)					
Recharge rate	ft/d	1	0.001	0.00103	Initially set to 10% of precipitation, then adjusted during calibration.
		2	NA	8.08×10^{-6}	Zone covering site areas added during calibration. Value adjusted during calibration.
		3	NA	0	Assumption of 0 recharge through impervious cover, including buildings, roadways, parking lots, and other features that appeared to be impervious on aerial photographs.

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Table 3
Initial and Post-Calibration Model Parameters

Parameter	Units	Layer or Zone	Initial Value	Final Value	Basis of Value/Comments
General Head Boundary (Layer 3 only)					
Head	ft	3	NA	578	Professional judgment. Adjusted during calibration.
Distance to head boundary	ft	3	NA	15	Set equal to thickness of Layer 3.
Hydraulic conductivity	ft/d	3	0.1	1.5×10^{-5}	Professional judgment. Adjusted during calibration.
Saturated thickness of unit	ft	3	NA	15	Set equal to distance to head boundary.
Drain Boundary (Layer 1 only)					
Bottom elevation	ft msl	1	564 to 607	563 to 607	
Sediment thickness	ft	1	NA	1	
Sediment hydraulic conductivity	ft/d	1	0.1	0.01 to 10	Value adjusted during calibration.
Lake Boundary (Layer 1 only)					
Initial Lake stage elevation	ft msl	1	564 to 607	563 to 607	
Minimum stage elevation	ft msl	1	NA	1	
Maximum stage elevation	ft msl	1	0.1	0.1	Value adjusted during calibration.
Runoff into lake	NA	NA	NA	NA	
Withdrawal from lake	NA	NA	NA	NA	
Sediment hydraulic conductivity	ft/d	1	0.1	0.001	Value adjusted during calibration.
Lakebed thickness	ft	1	NA	1	

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Table 4
Effect of Barrier Walls on Individual Well Groundwater Elevations

Well	X (ft)	Y (ft)	Model Layer	Simulated GW Elevation (ft msl) Before Wall	After Wall	Change
EW-8	1725569	524315	1	570.86	571.01	0.16
GW-10A	1726519	525942	1	568.23	571.09	2.86
GW-10B	1726516	525937	2	568.35	571.09	2.74
GW-11A	1726718	526196	1	567.64	571.11	3.47
GW-11B	1726715	526192	2	567.63	571.11	3.48
GW-12A	1727979	527149	1	567.77	571.59	3.82
GW-12B	1727983	527148	2	567.64	571.59	3.95
GW-13A	1728032	527278	1	567.19	566.51	-0.67
GW-13B	1728027	527276	2	567.11	566.51	-0.60
GW-14A	1728173	527345	1	567.16	566.46	-0.70
GW-14B	1728168	527343	2	567.09	566.46	-0.62
GW-15A	1729240	527194	1	570.96	571.24	0.28
GW-15B	1729234	527196	2	570.84	571.24	0.40
GW-16A	1729352	527585	1	569.51	569.65	0.13
GW-16B	1729346	527587	2	569.34	569.59	0.25
GW-17A	1729454	528057	2	566.89	566.92	0.04
GW-17B	1729456	528061	2	566.89	566.92	0.04
GW-1A	1727055	524557	1	573.66	574.15	0.49
GW-1B	1727053	524553	2	573.66	574.15	0.49
GW-2A	1726195	524095	1	572.84	573.23	0.39
GW-2B	1726191	524098	2	572.84	573.23	0.39
GW-3A	1725688	524646	1	570.18	570.33	0.16
GW-3B	1725690	524651	2	570.16	570.33	0.17
GW-4A	1725929	525095	1	569.79	570.00	0.21
GW-4B	1725932	525100	2	569.77	570.00	0.23
GW-5A	1726799	525518	2	570.97	571.54	0.57
GW-5B	1726800	525513	2	571.04	571.52	0.48
GW-6A	1728364	526888	1	570.33	571.45	1.12
GW-6B	1728368	526884	2	570.21	571.45	1.24
GW-7A	1728535	527062	1	570.08	573.82	3.74
GW-7B	1728530	527063	2	569.97	573.82	3.86
GW-8A	1729803	527738	1	569.56	569.66	0.10
GW-8B	1729803	527744	2	569.35	569.56	0.21
GW-9A	1726430	525677	1	569.32	571.11	1.79
GW-9B	1726427	525673	2	569.32	571.11	1.80
MW-17A	1726701	523865	2	574.59	575.14	0.55
MW-18A	1725987	524125	1	572.33	572.63	0.30
MW-19A	1725679	524615	1	570.30	570.47	0.16
MW-1A	1727205	523727	1	575.82	576.39	0.57
MW-23A	1725558	524298	1	570.86	571.01	0.16
MW-3A	1725627	524444	1	570.58	570.74	0.16

Note: Coordinates are State Plane, West Virginia South, NAD 83, U.S. survey feet.

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Table 5
Estimated Pumping Rates Required to Maintain an Inward Hydraulic Gradient Across Barrier Walls

Well	Model Grid Location		Pumping Rate		Pumping Well Radius[1]	Screen Bottom Elevation[2]	Simulated Head In Grid Cell	Calculated Head at Well[3]	Minimum Allowed Head[4]	Water Above Min. Head	Feasibility Indication[5]
	Row	Col	(ft^3/d)	(gpm)	(ft)	(ft MSL)	(ft MSL)	(ft MSL)	(ft MSL)	(ft)	
WTA West											
GW-12A	139	148	30	0.16	0.08	551.5	564.5	561.4	556.5	4.9	OK
WTA-New-1	134	156	30	0.16	0.08	550.0	565.1	562.2	555.0	7.2	OK
Total WTA West				0.32							
WTA East											
GW-7A	143	170	40	0.21	0.08	552.9	566.9	563.4	557.9	5.5	OK
Total WTA East				0.21							
PDA											
GW-9A	198	86	65	0.34	0.08	552.0	565.1	557.5	557.0	0.5	OK
GW-10A	187	89	55	0.29	0.08	551.8	563.8	556.7	556.8	-0.1	LOW
GW-11A	177	97	40	0.21	0.08	554.5	564.3	560.0	559.5	0.5	OK
PDA-New_1	182	93	65	0.34	0.17	550.0	563.1	555.3	555.0	0.3	OK
Total PDA				1.18							
PA											
GW-1A	243	111	60	0.31	0.08	550.0	570.3	566.0	555.0	11.0	OK
PA-New_1	233	87	70	0.36	0.17	550.0	567.5	562.3	555.0	7.3	OK
Total PA				0.67							

Notes:
1. All new wells are assumed to be 4 inches in diameter.
2. New wells are assumed to be screened down to an elevation of 550 feet MSL.
3. The Theim equation was used to calculate the actual head at production wells as described by Anderson and Woessner (2002) page 149.
4. A 5-foot minimum saturated thickness was assumed to restrict drawdown at all locations.
5. "OK" means predicted saturated thickness in the pumping well is above the minimum. "LOW" indicates that the predicted saturated thickness is below the minimum.

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Table 6
Effect of Kanawha River Stage on Individual Well Groundwater Elevations After Construction of Barrier Walls

Well	X (ft)	Y (ft)	Model Layer	Current 566 ft msl	Simulated GW Elevation and Change at River Stage Indicated (ft msl): +2 ft (568 ft msl) Head	+2 ft (568 ft msl) Change	+5 ft (571 ft msl) Head	+5 ft (571 ft msl) Change	-2 ft (564 ft msl) Head	-2 ft (564 ft msl) Change	-5 ft (561 ft msl) Head	-5 ft (561 ft msl) Change
EW-8	524315	1725569	1	570.86	572.43	1.57	575.08	4.22	569.00	-1.86	566.29	-4.57
GW-1A	524557	1727055	1	568.23	577.17	8.94	578.92	10.69	575.00	6.77	573.12	4.89
GW-1B	524553	1727053	2	568.35	577.17	8.82	578.92	10.57	575.00	6.65	573.12	4.77
GW-2A	524095	1726195	1	567.64	574.97	7.33	577.20	9.56	572.00	4.36	569.79	2.15
GW-2B	524098	1726191	2	567.63	574.97	7.34	577.20	9.57	572.00	4.37	569.79	2.16
GW-3A	524646	1725688	1	567.77	571.54	3.77	574.23	6.46	568.00	0.23	565.30	-2.47
GW-3B	524651	1725690	2	567.64	571.54	3.90	574.23	6.59	568.00	0.36	565.30	-2.34
GW-4A	525095	1725929	1	567.19	571.25	4.06	573.93	6.74	568.00	0.81	565.02	-2.17
GW-4B	525100	1725932	2	567.11	571.25	4.14	573.93	6.82	568.00	0.89	565.02	-2.09
GW-5A	525518	1726799	2	567.16	575.14	7.98	577.32	10.16	572.00	4.84	570.11	2.95
GW-5B	525513	1726800	2	567.09	575.13	8.04	577.30	10.21	572.00	4.91	570.10	3.01
GW-6A	526888	1728364	1	570.96	575.93	4.97	578.12	7.16	573.00	2.04	570.85	-0.11
GW-6B	526884	1728368	2	570.84	575.93	5.09	578.12	7.28	573.00	2.16	570.85	0.01
GW-7A	527062	1728535	1	569.51	575.93	6.42	578.12	8.61	573.00	3.49	570.85	1.34
GW-7B	527063	1728530	2	569.34	575.93	6.59	578.12	8.78	573.00	3.66	570.85	1.51
GW-8A	527738	1729803	1	566.89	571.24	4.35	573.54	6.65	568.00	1.11	565.89	-1.00
GW-8B	527744	1729803	2	566.89	571.15	4.26	573.47	6.58	568.00	1.11	565.75	-1.14
GW-9A	525677	1726429	1	573.66	574.87	1.21	577.08	3.42	572.00	-1.66	569.77	-3.89
GW-9B	525673	1726427	2	573.66	574.87	1.21	577.08	3.42	572.00	-1.66	569.77	-3.89
GW-10A	525942	1726519	1	572.84	574.81	1.97	577.02	4.18	572.00	-0.84	569.68	-3.16
GW-10B	525937	1726516	2	572.84	574.82	1.98	577.03	4.19	572.00	-0.84	569.71	-3.13
GW-11A	526196	1726718	1	570.18	574.72	4.54	576.94	6.76	572.00	1.82	569.57	-0.61
GW-11B	526192	1726715	2	570.16	574.72	4.56	576.94	6.78	572.00	1.84	569.57	-0.59
GW-12A	527149	1727979	1	569.79	575.50	5.71	577.75	7.96	573.00	3.21	570.28	0.49
GW-12B	527148	1727983	2	569.77	575.50	5.73	577.75	7.98	573.00	3.23	570.28	0.51
GW-13A	527278	1728032	1	570.97	568.54	-2.43	571.49	0.52	565.00	-5.97	561.65	-9.32
GW-13B	527276	1728027	2	571.04	568.54	-2.50	571.49	0.45	565.00	-6.04	561.65	-9.39
GW-14A	527345	1728173	1	570.33	568.52	-1.81	571.46	1.13	565.00	-5.33	561.62	-8.71
GW-14B	527343	1728168	2	570.21	568.52	-1.69	571.46	1.25	565.00	-5.21	561.62	-8.59
GW-15A	527194	1729240	1	570.08	572.66	2.58	574.84	4.76	570.00	-0.08	567.62	-2.46
GW-15B	527196	1729234	2	569.97	572.66	2.69	574.84	4.87	570.00	0.03	567.62	-2.35
GW-16A	527585	1729352	1	569.56	571.33	1.77	573.74	4.18	568.00	-1.56	565.69	-3.87
GW-16B	527587	1729346	2	569.35	571.29	1.94	573.72	4.37	568.00	-1.35	565.62	-3.73
GW-17A	528057	1729454	2	569.32	568.93	-0.39	571.77	2.45	565.00	-4.32	562.35	-6.97
GW-17B	528061	1729456	2	569.32	568.93	-0.39	571.77	2.45	565.00	-4.32	562.35	-6.97
MW-1A	523727	1727205	1	574.59	577.43	2.84	579.06	4.47	575.00	0.41	573.63	-0.96
MW-3A	524444	1725627	1	572.33	572.07	-0.26	574.72	2.39	569.00	-3.33	565.92	-6.41
MW-17A	523865	1726701	2	570.30	576.62	6.32	578.52	8.22	574.00	3.70	572.20	1.90
MW-18A	524125	1725987	1	575.82	574.33	-1.49	576.68	0.86	571.00	-4.82	568.87	-6.95
MW-19A	524615	1725679	1	570.86	571.73	0.87	574.40	3.54	568.00	-2.86	565.52	-5.34
MW-23A	524298	1725558	1	570.58	572.43	1.85	575.08	4.50	569.00	-1.58	566.29	-4.29

Note: Coordinates are State Plane, West Virginia South, NAD 83, U.S. survey feet.

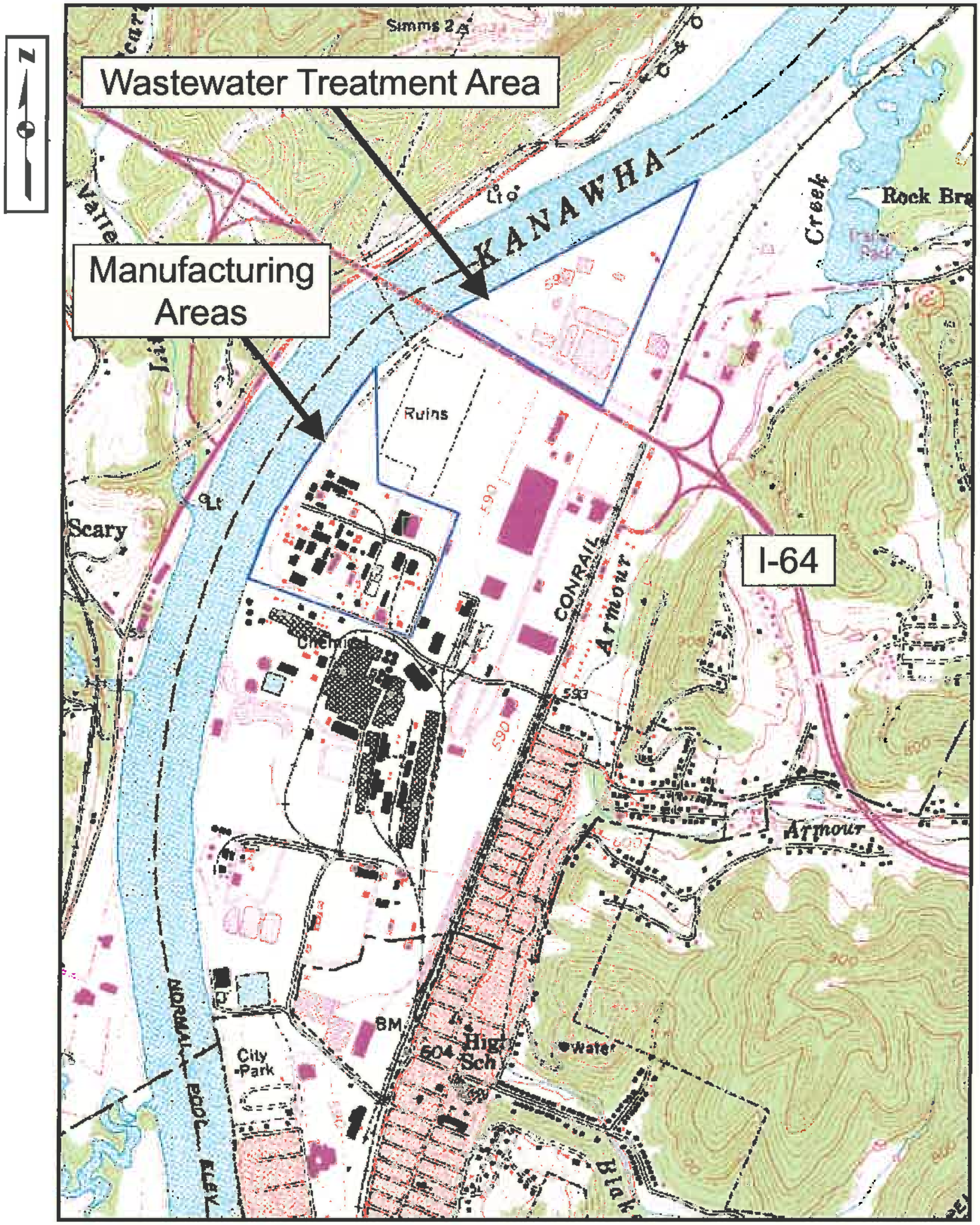
FIGURES

GSI ENVIRONMENTAL

Figures

GSI Job No. G-3559
9 September 2011

Figure 1
Site Location

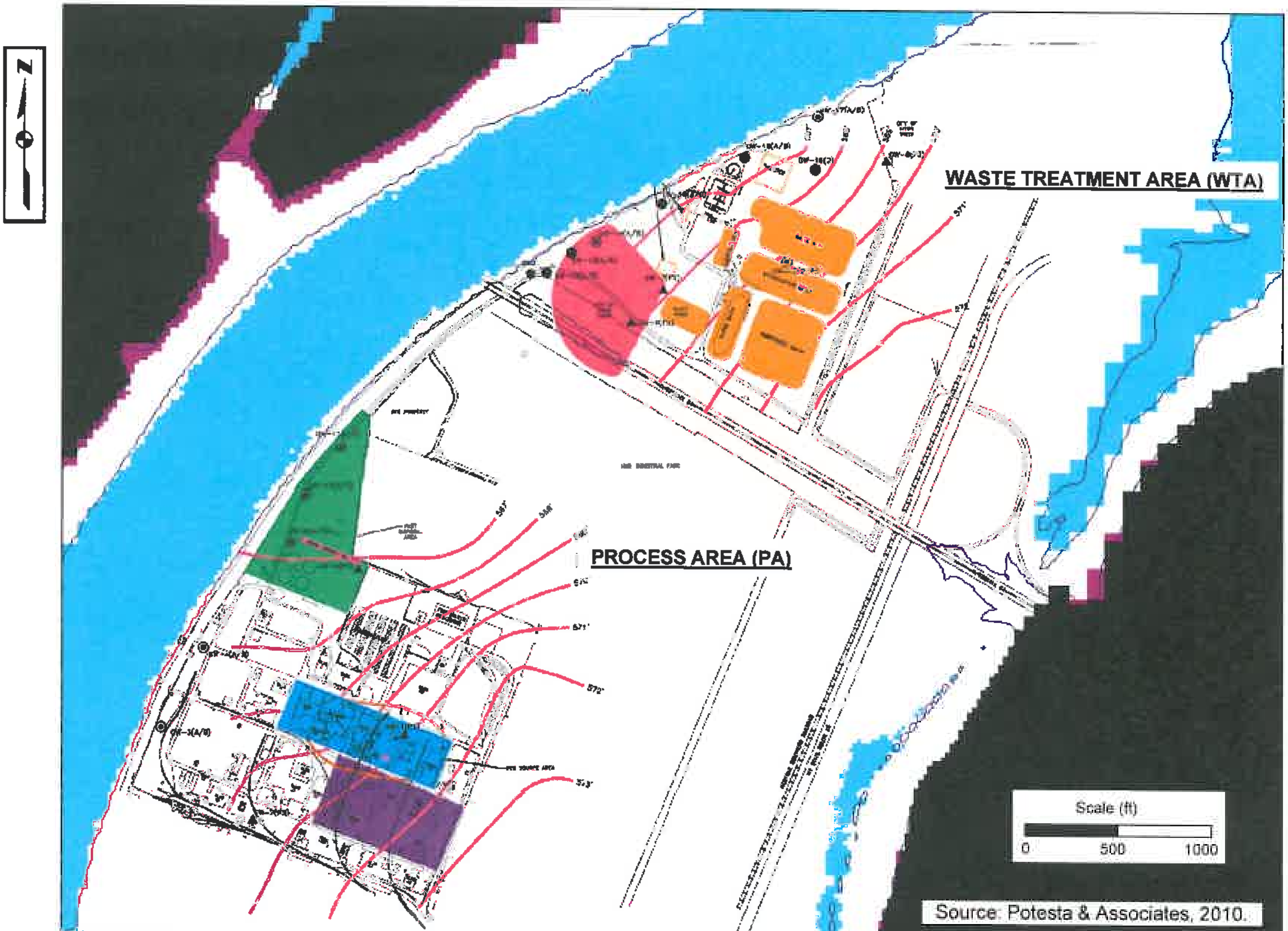


Source: USGS St. Albans, WV Quadrangle
7.5 minute series topographic map, photorevised 1976.

Scale (ft)
0 1000 2000

GSI Job No. G-3559
9 September 2011

Figure 2
Zone A and B Potentiometric Surface Interpolated from Measured Water Levels



GSI Job No. G-3559
9 September 2011

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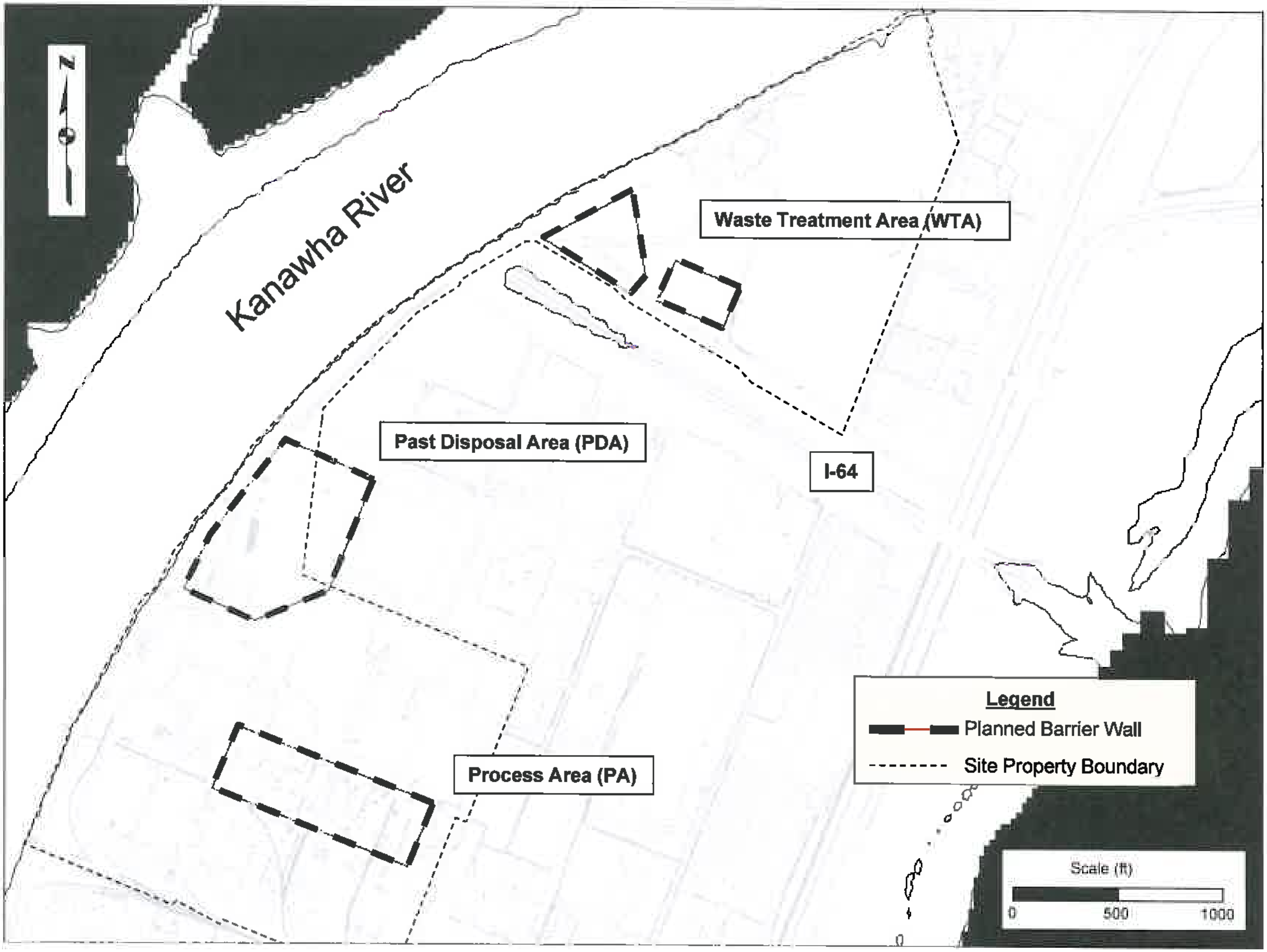
Figure 3
Planned Interim Measures: Barrier Wall Locations



GSI Job No. G-3559
9 September 2011

Figure 4
Locations of Bedrock Pumping Tests

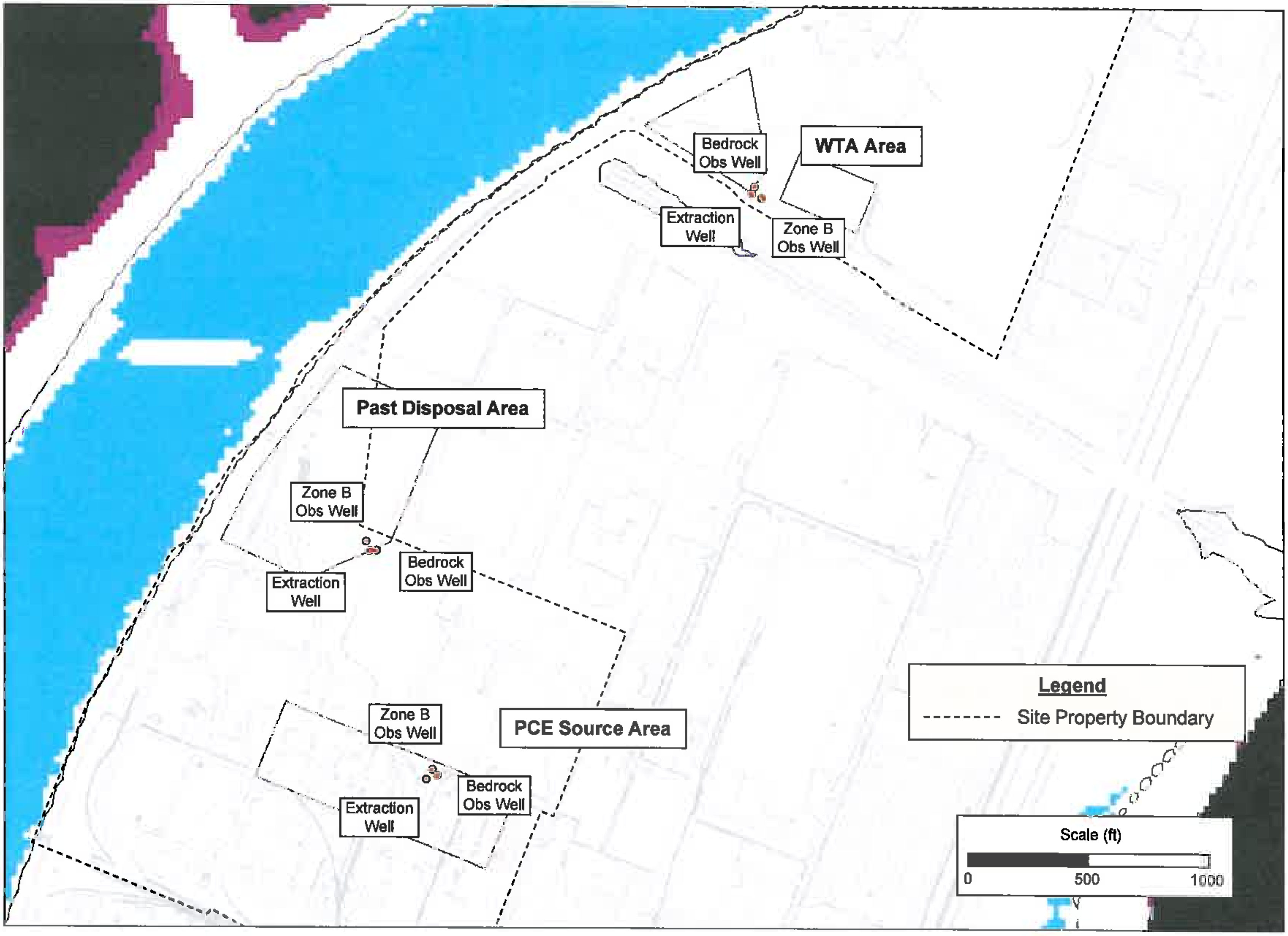


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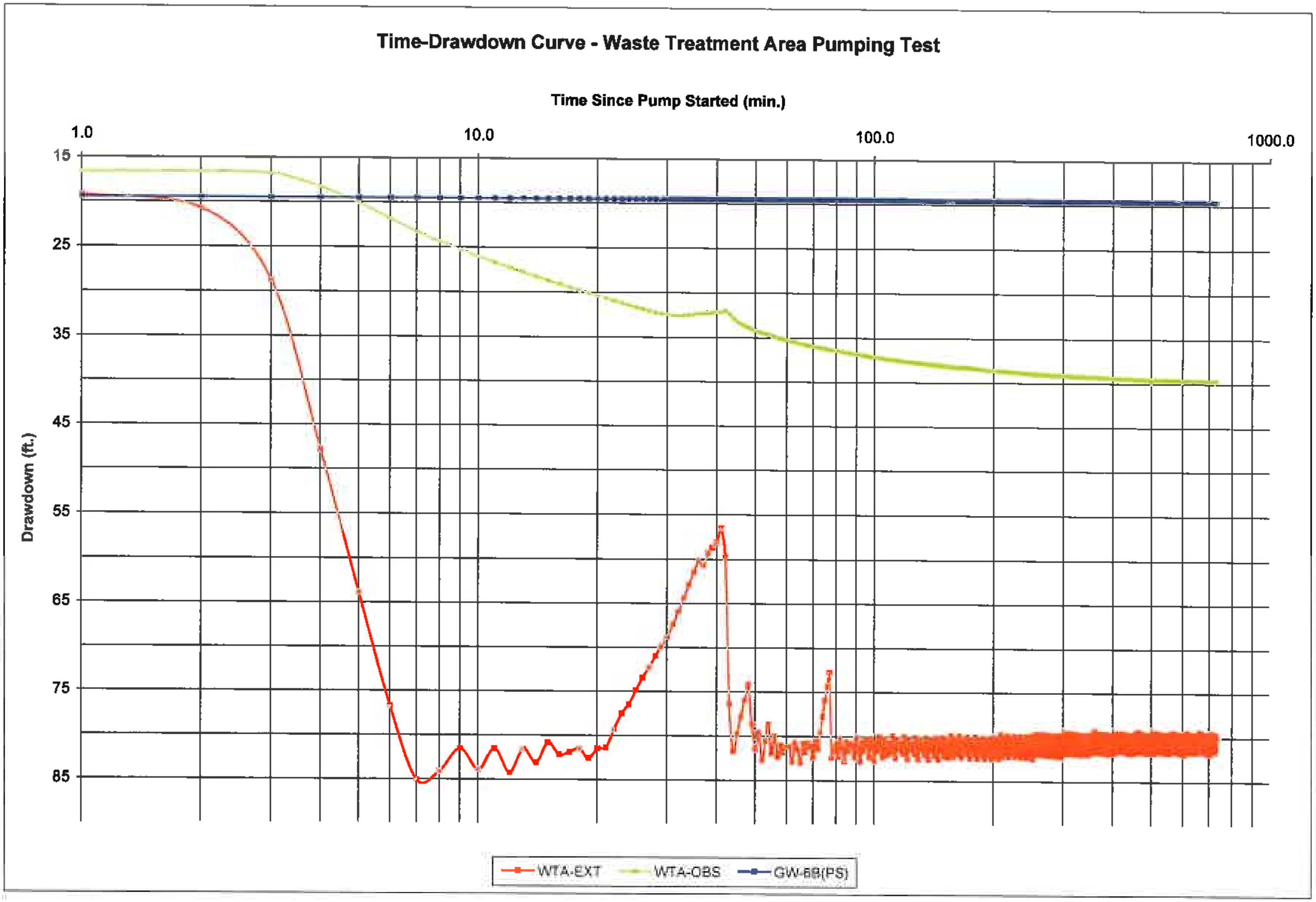
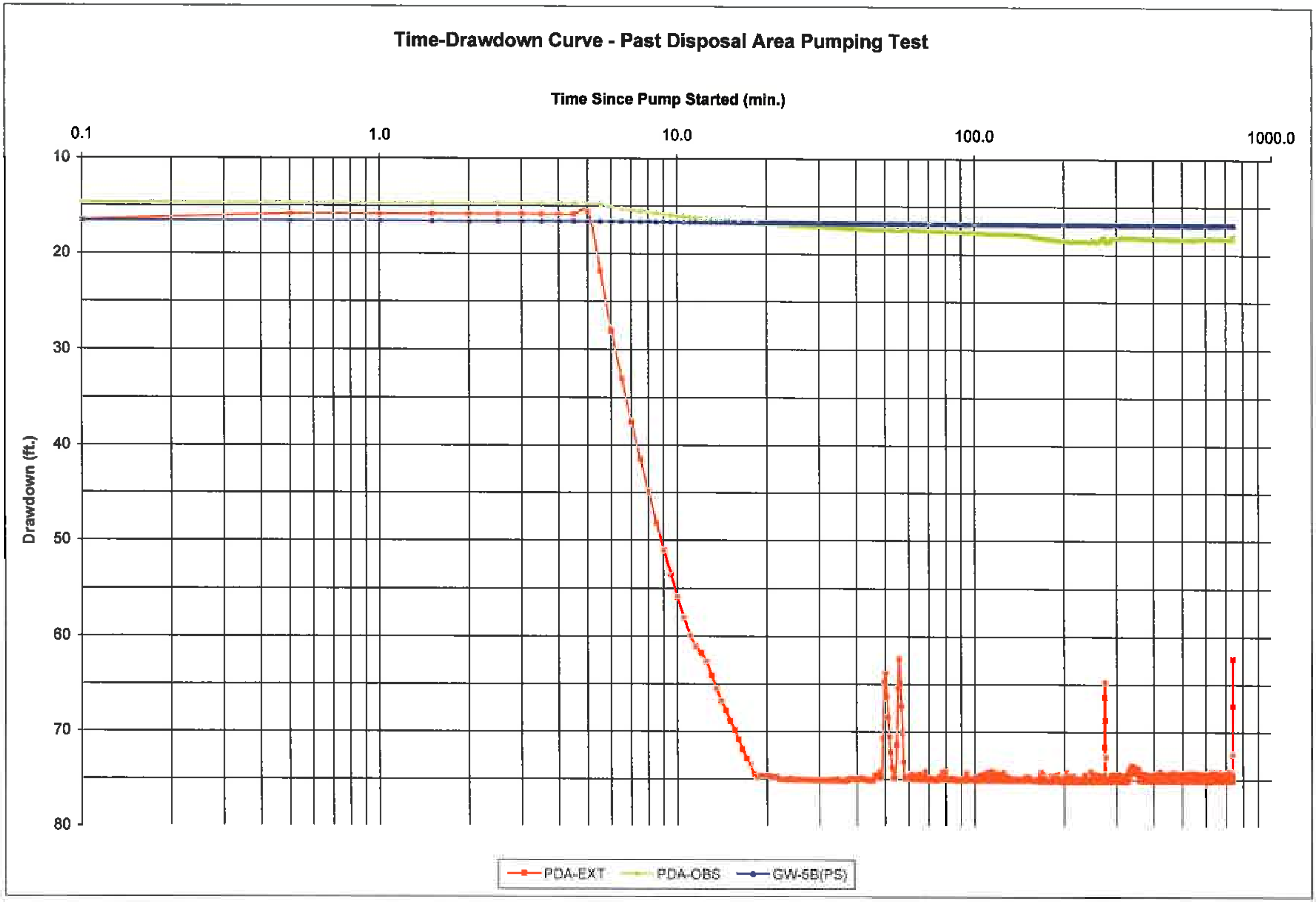
Figure 5



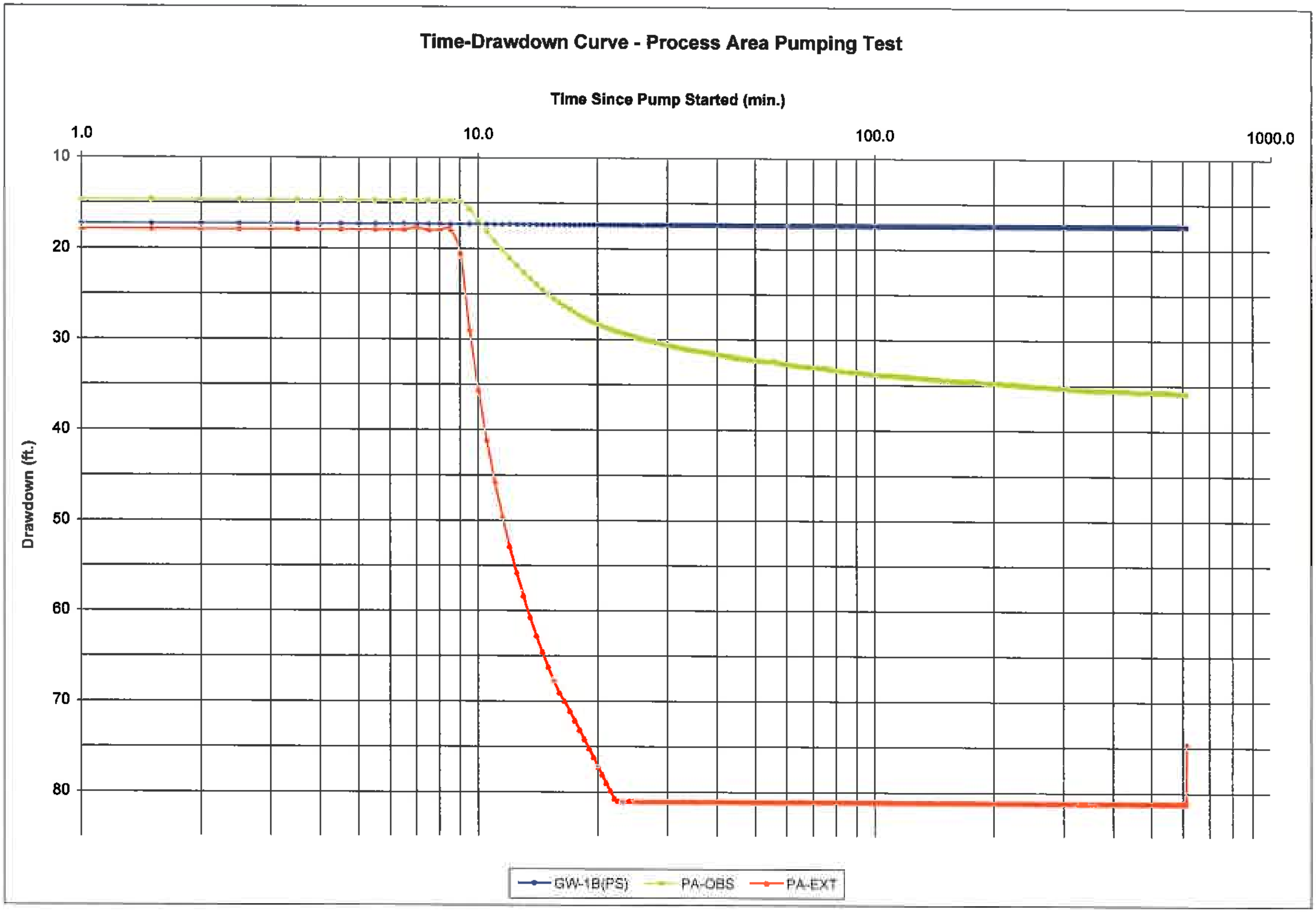
Reproduced from plot created by Potesta and Associates, 2010.

Figure 6



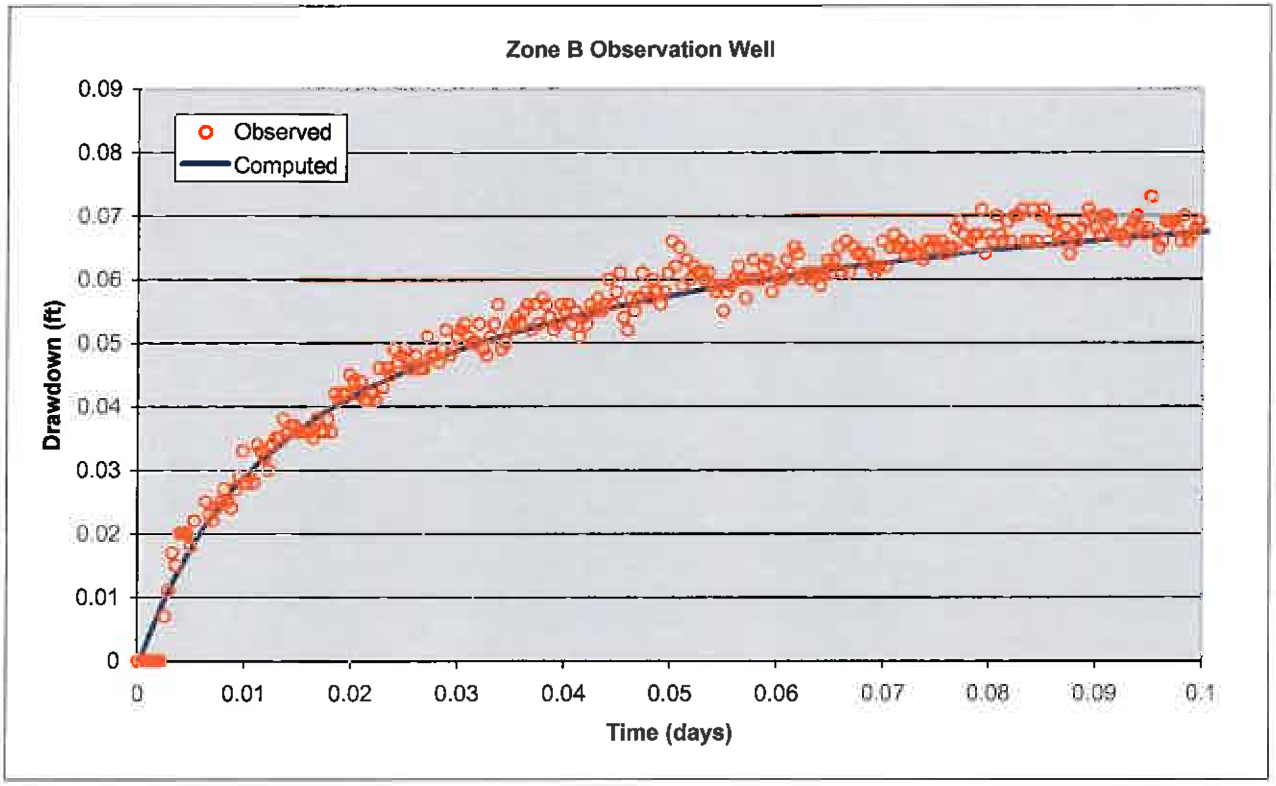
Reproduced from plot created by Potesta and Associates, 2010.

Figure 7



Reproduced from plot created by Potesta and Associates, 2010.

Figure 8
Comparison of Simulated to Measured Pumping Test Drawdown at the WTA



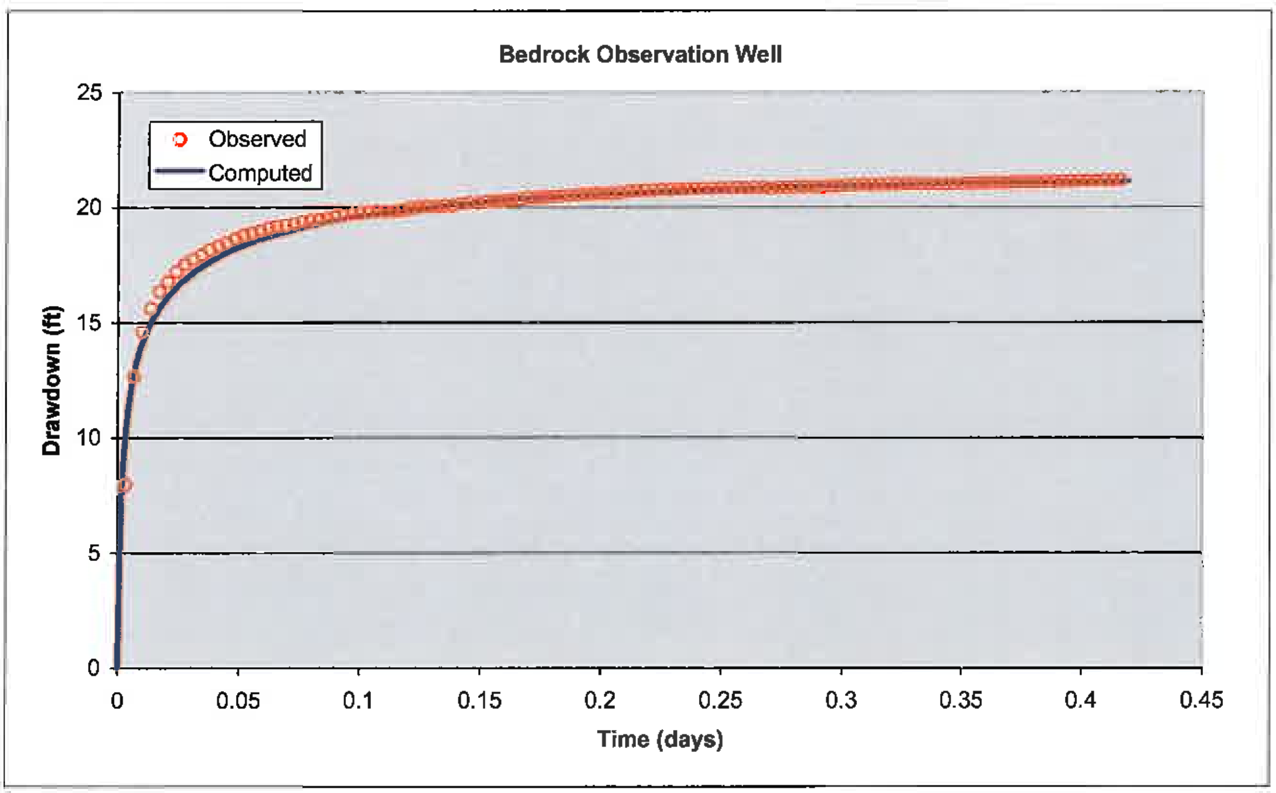
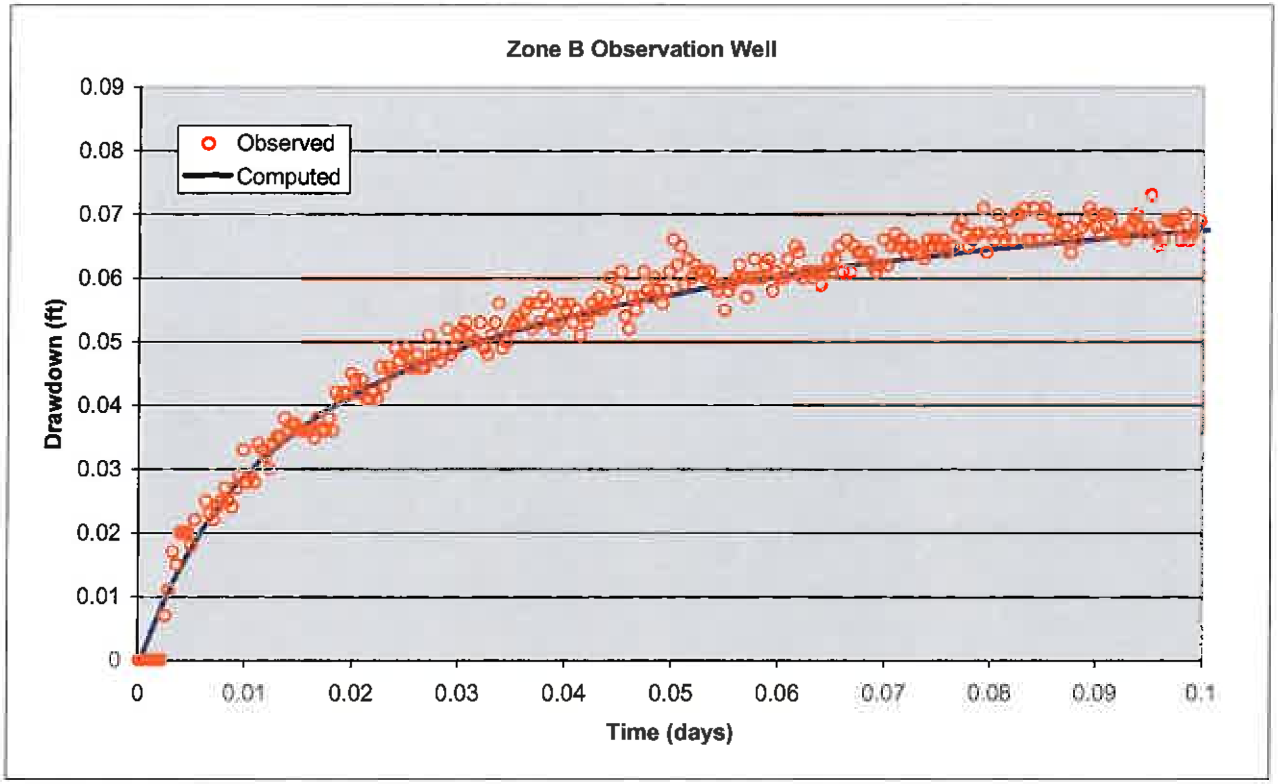
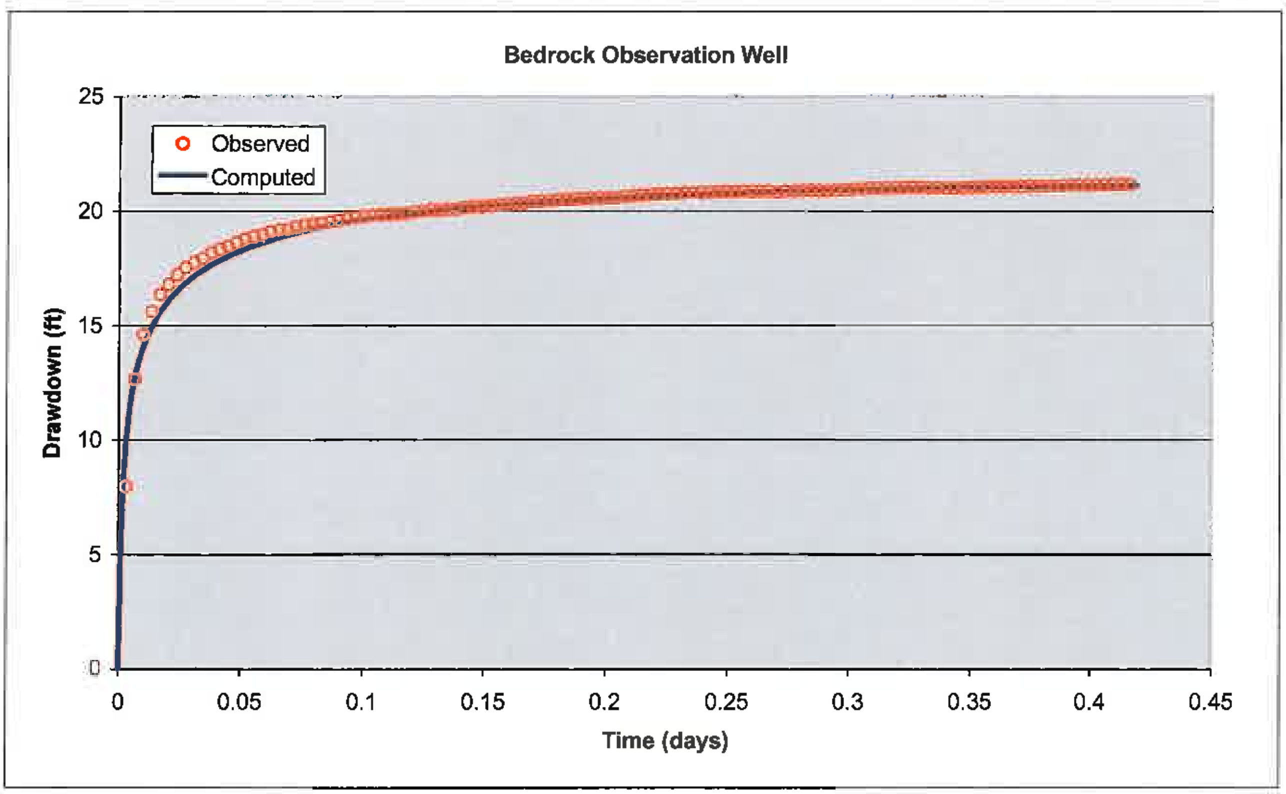


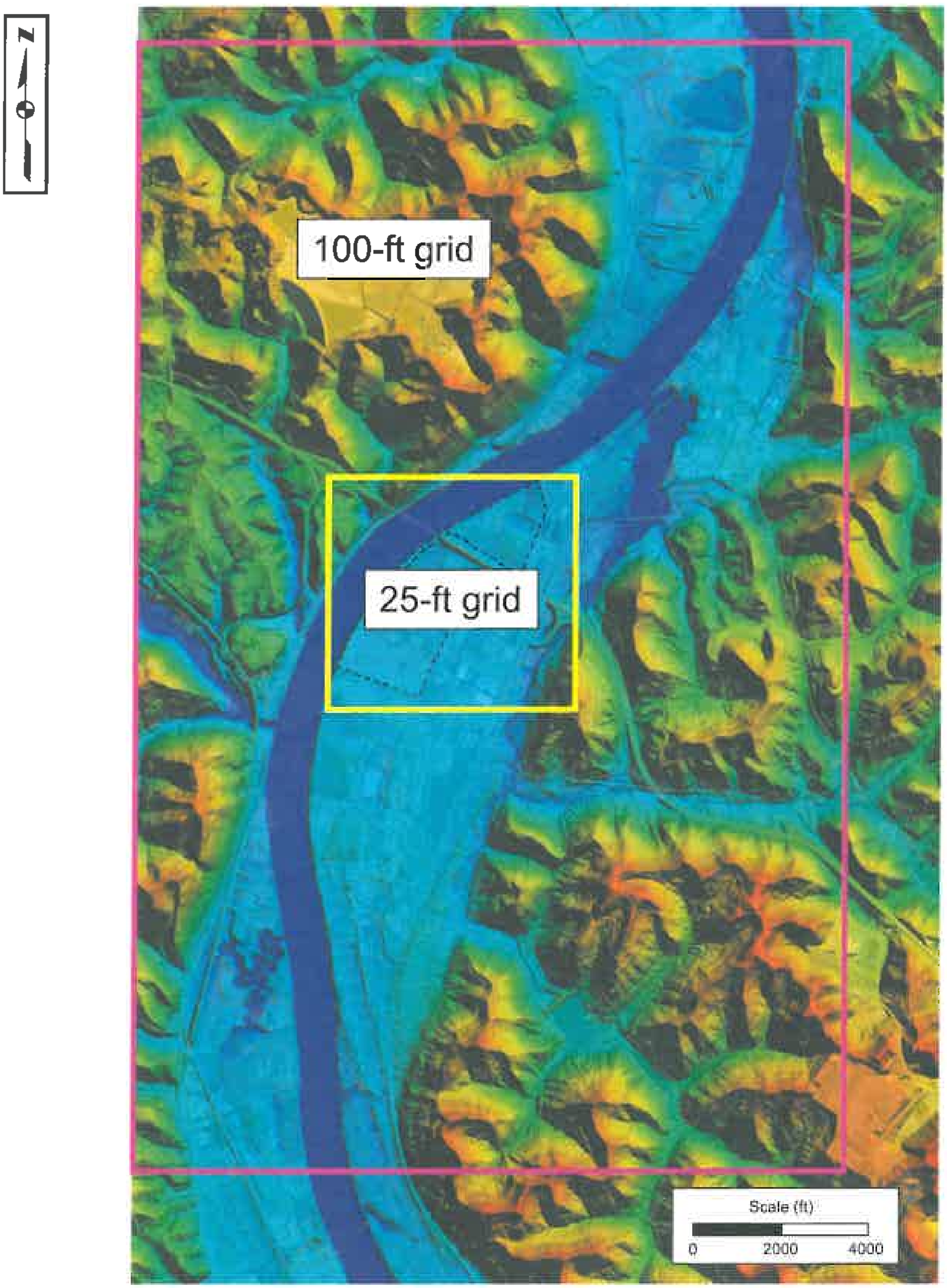
Figure 9
Comparison of Simulated to Measured Pumping Test Drawdown at the PA





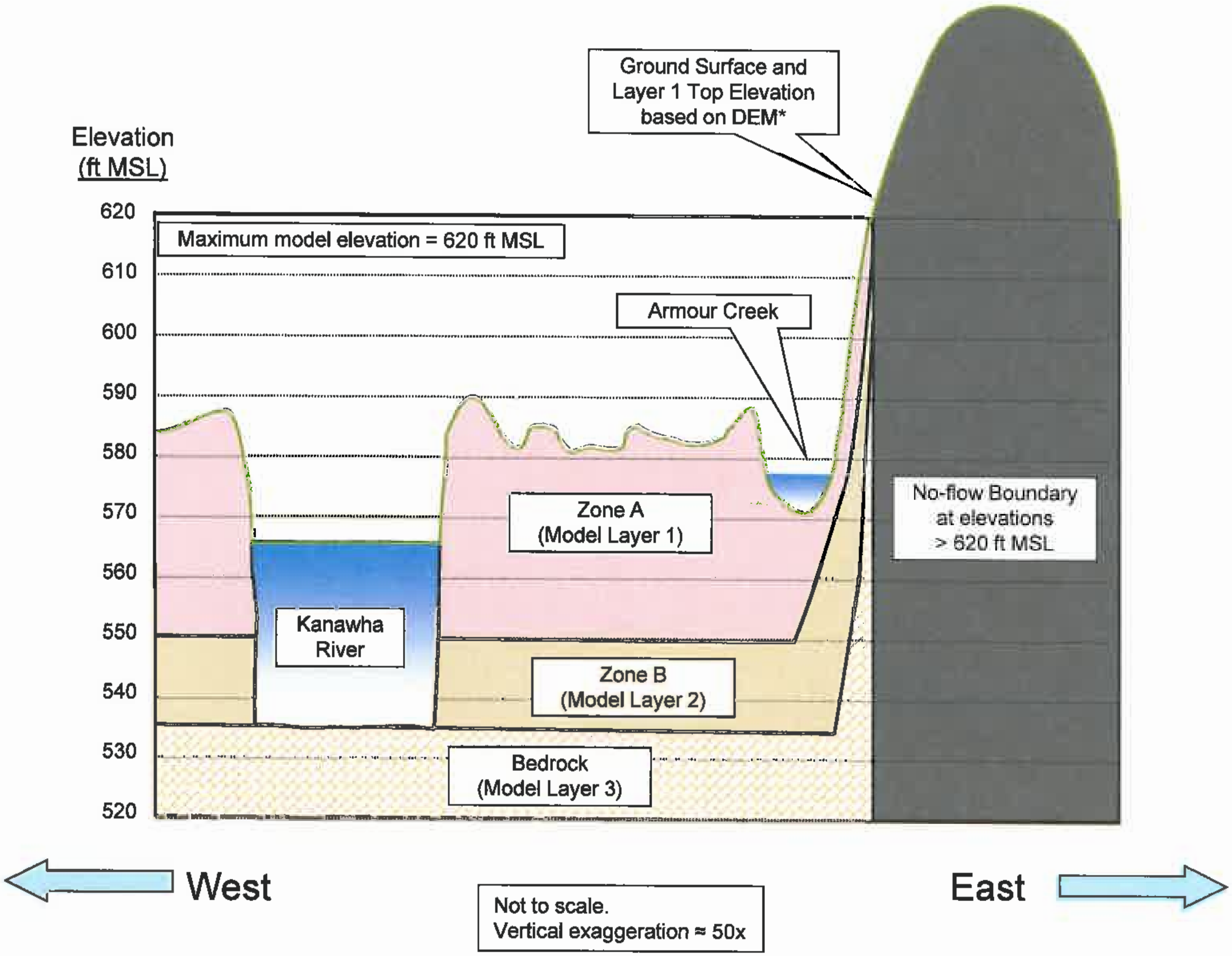
GSI Job No. G-3559
9 September 2011

Figure 10
Horizontal Model Domain Extent and Discretization



GSI Job No. G-3559
9 September 2011

Figure 11
Representative Conceptual Model Cross Section



* DEM = digital elevation model.

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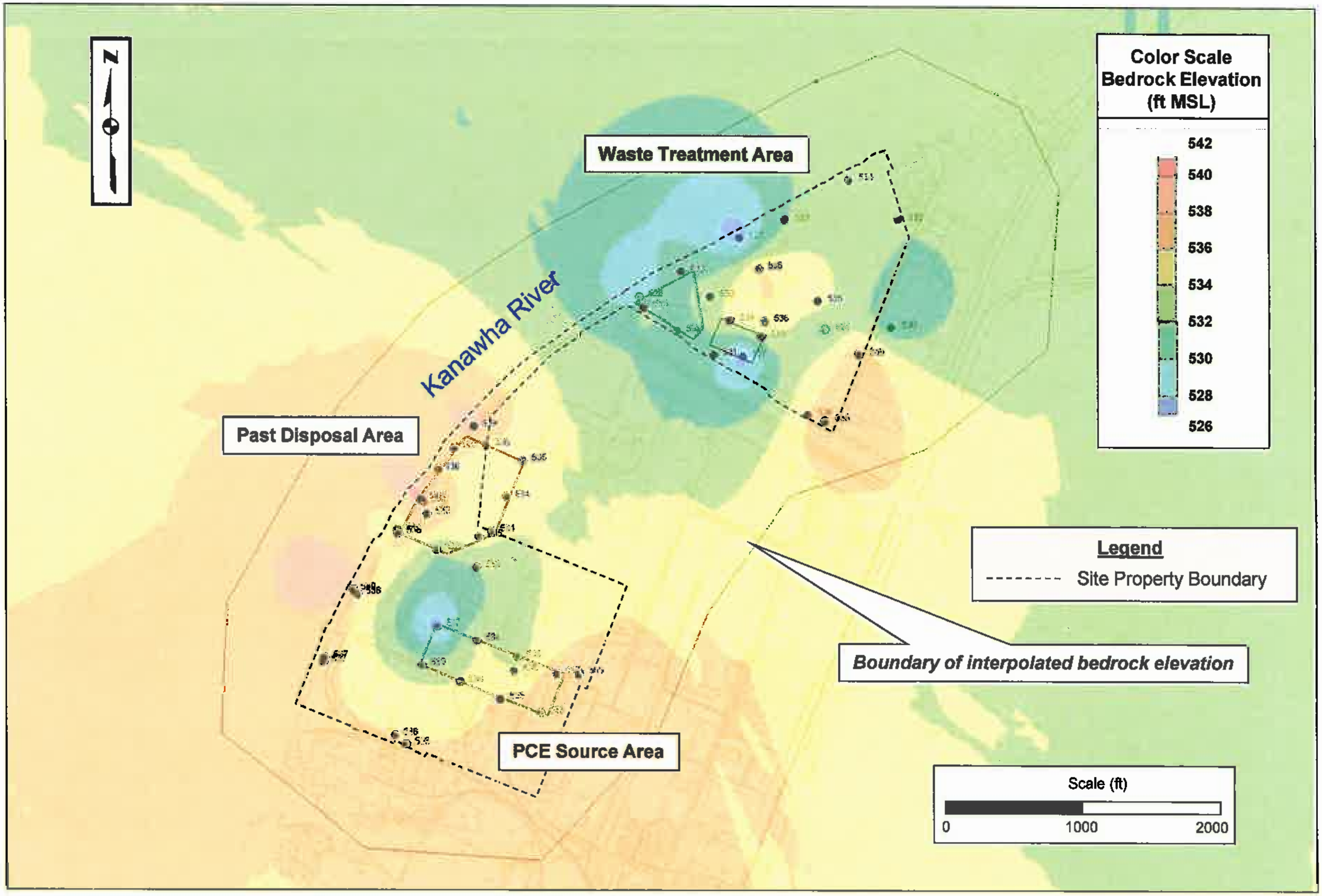
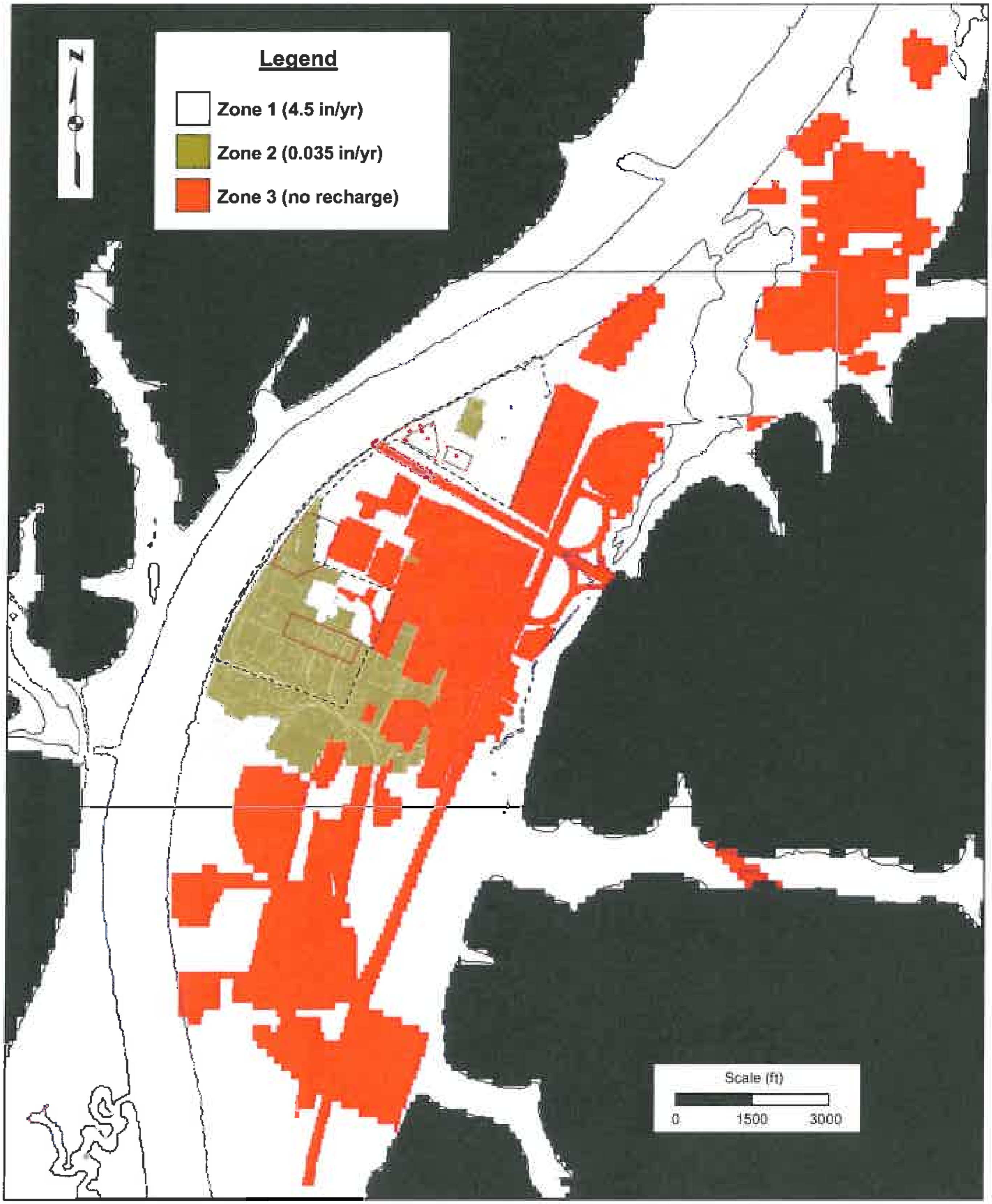
9 September 2011

Figure 12
Interpolated Bedrock Elevations in the Site Area



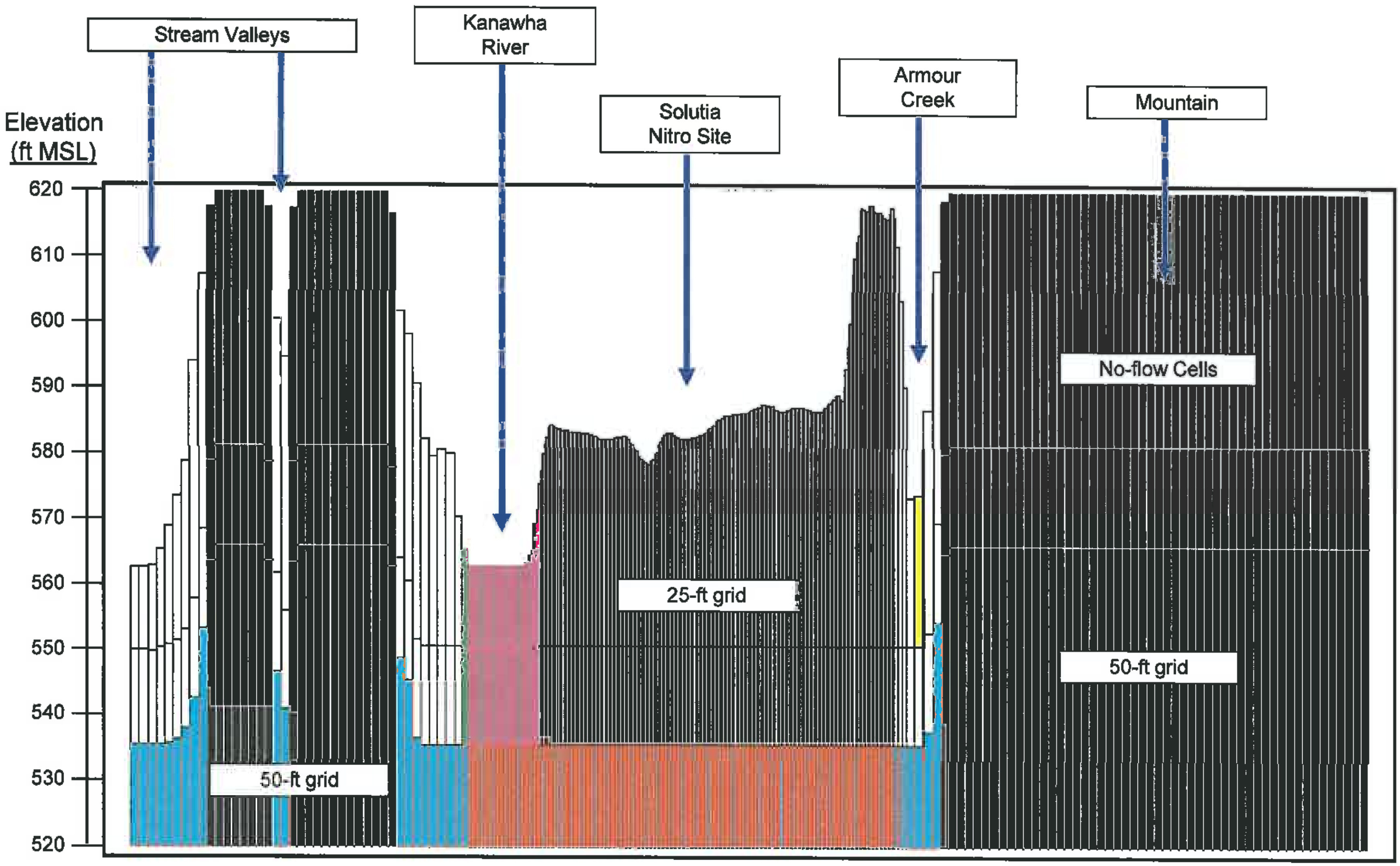
GSI ENVIRONMENTAL

Figure 13
Model Recharge Zones and Recharge Rates



GSI Job No. G-3559
9 September 2011

Figure 14
Vertical Model Domain Discretization – Representative Cross Section at Model Row 200



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9 September 2011

Figure 15
Layer 1 (Zone A) Boundary Conditions

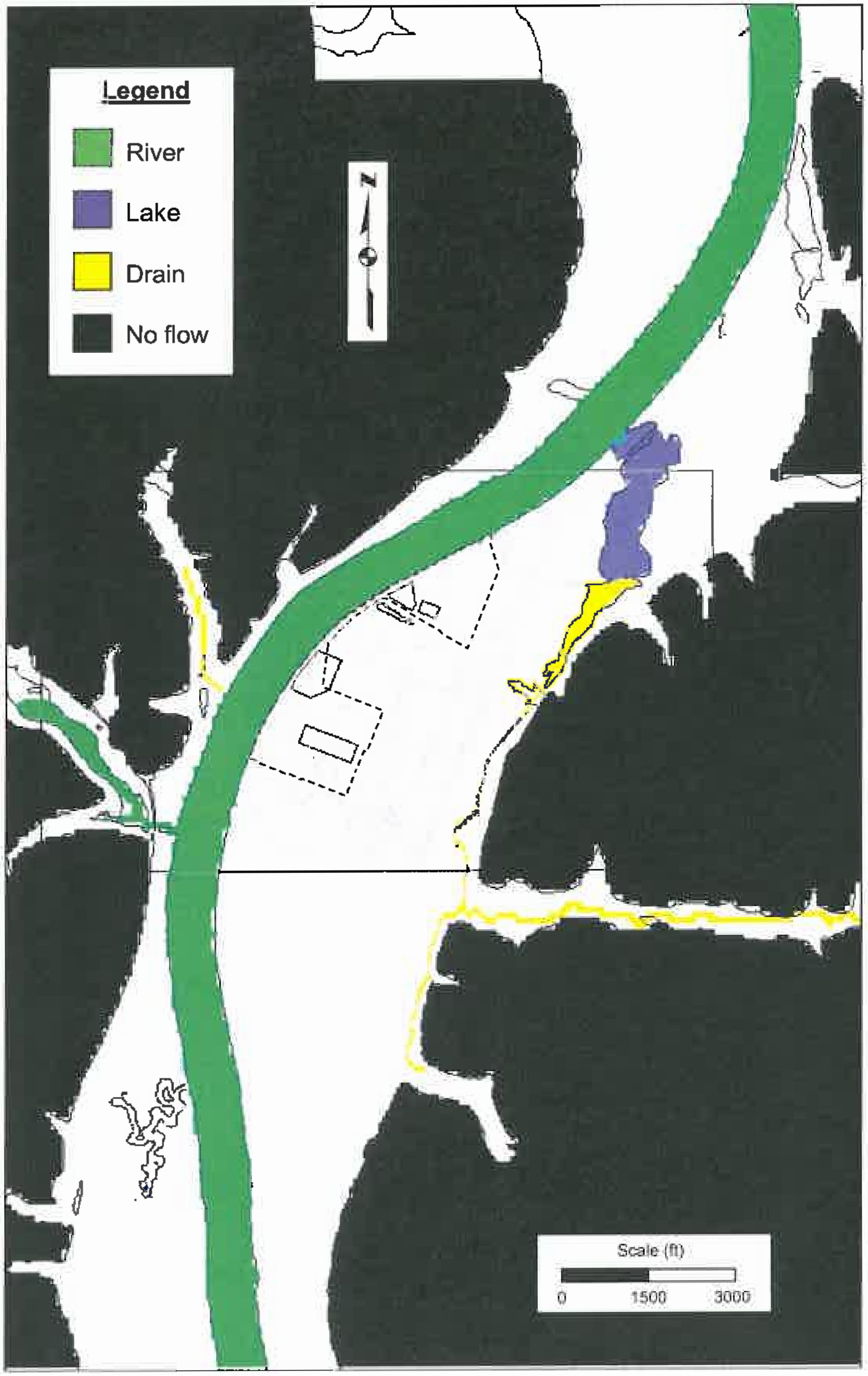

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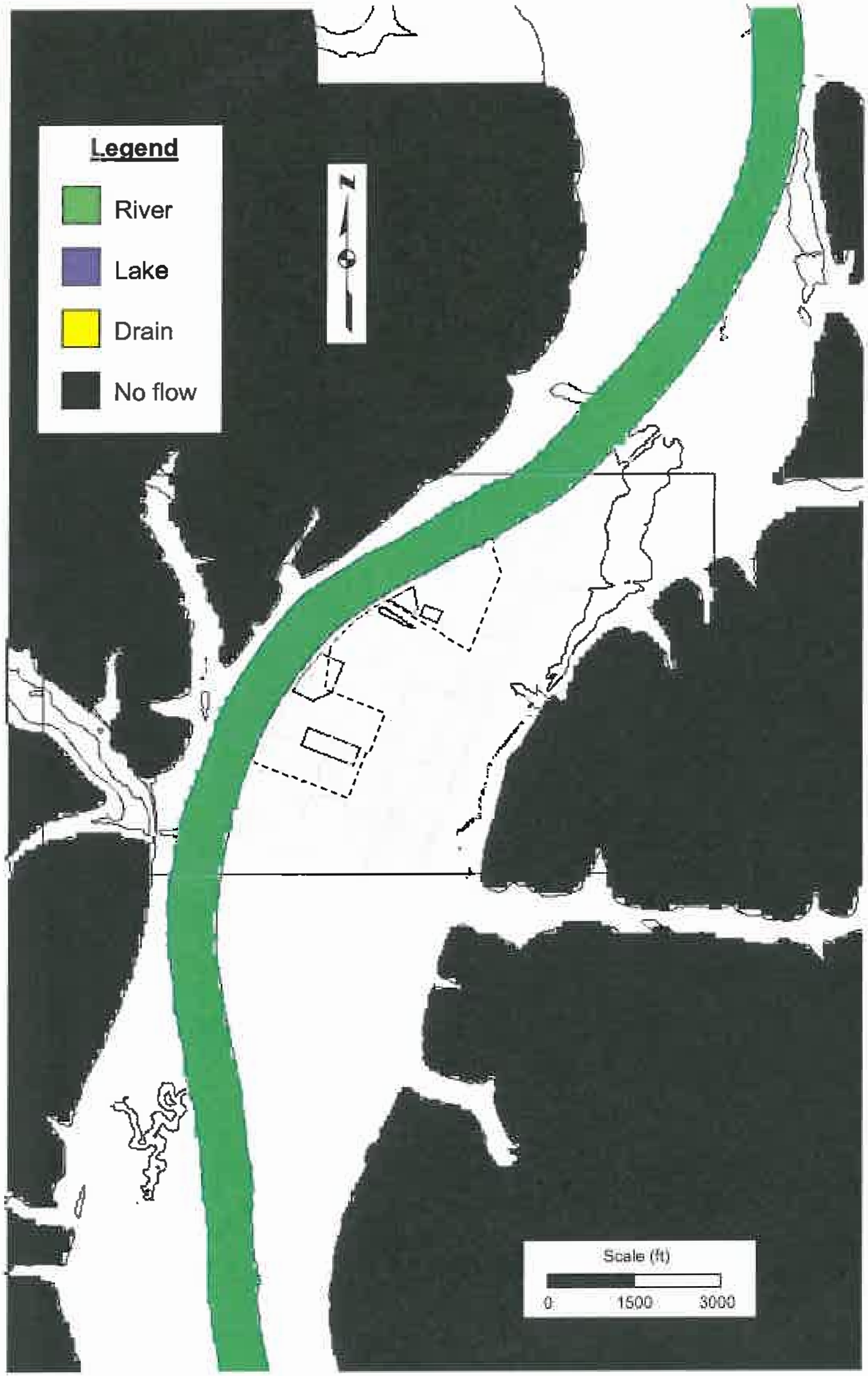
Figure 16
Layer 2 (Zone B) Boundary Conditions



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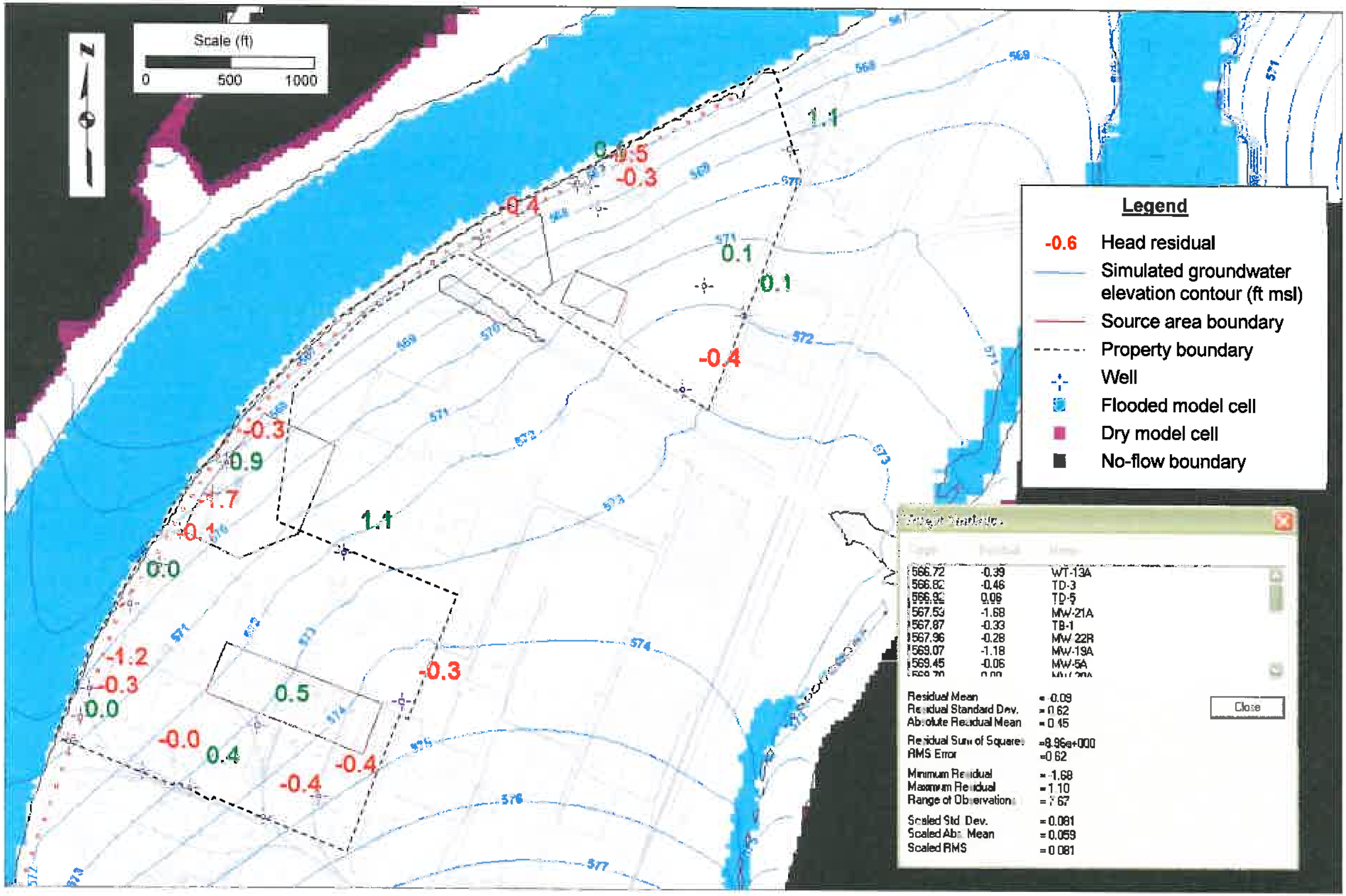
9 September 2011

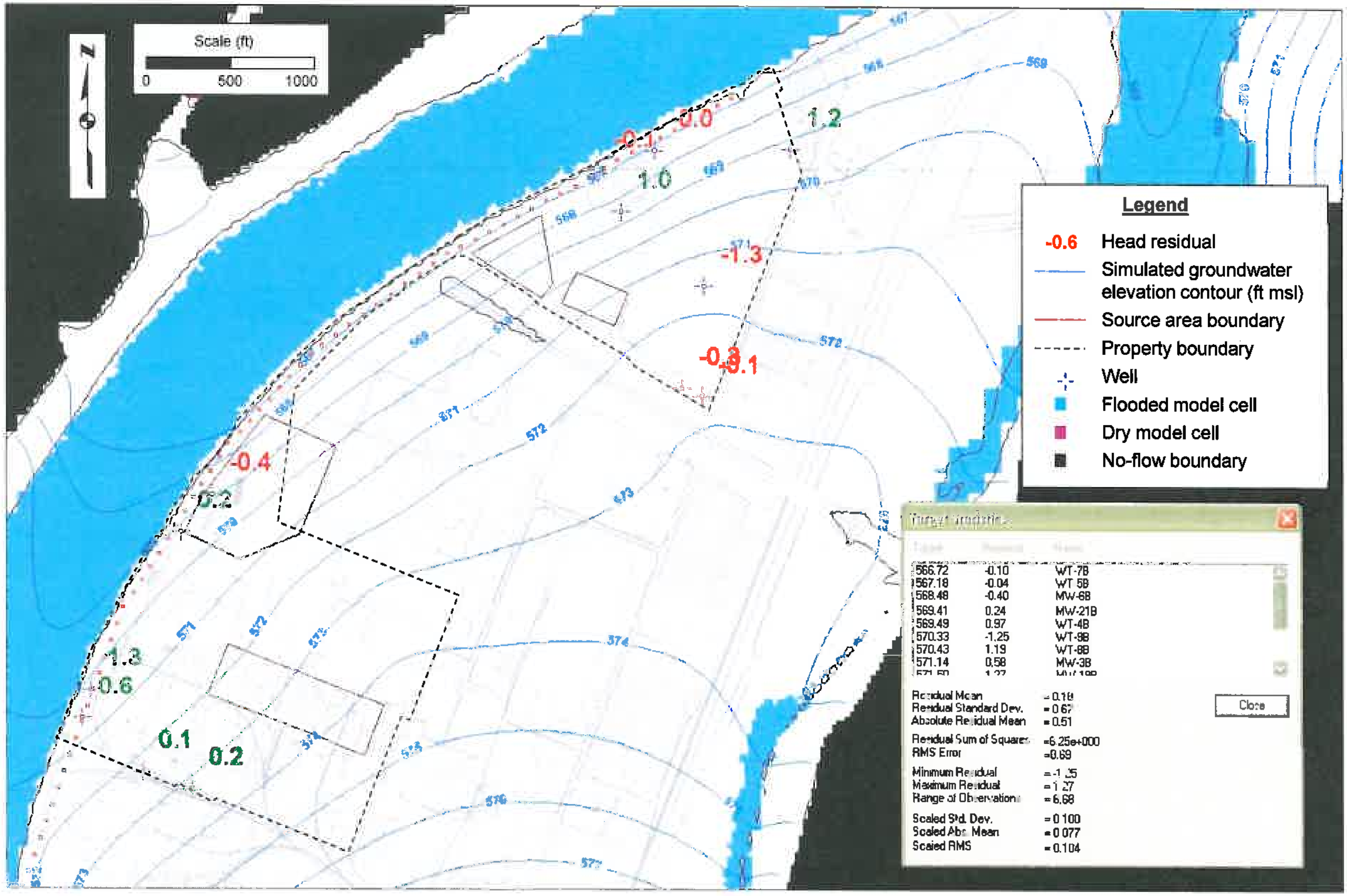
GSI ENVIRONMENTAL

Figure 17
Simulated Potentiometric Surface and Calibration Residual in Zone A



GSI Job No. G-3559
9 September 2011

Figure 18
Simulated Potentiometric Surface and Calibration Residual in Zone B



GSI Job No. G-3559
9 September 2011

GSI ENVIRONMENTAL

Figure 19
Simulated Potentiometric Surface and Calibration Residual in the Bedrock

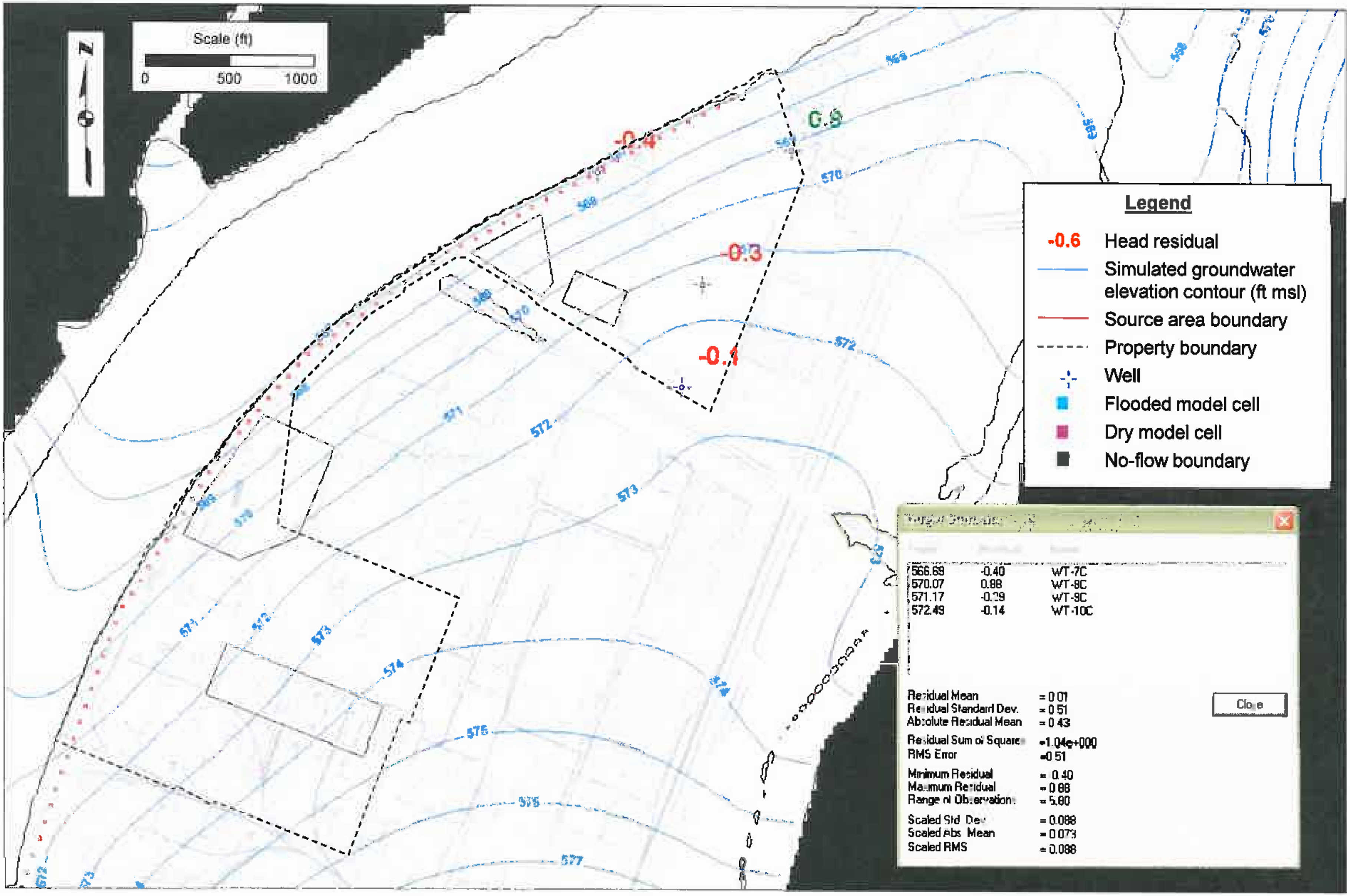
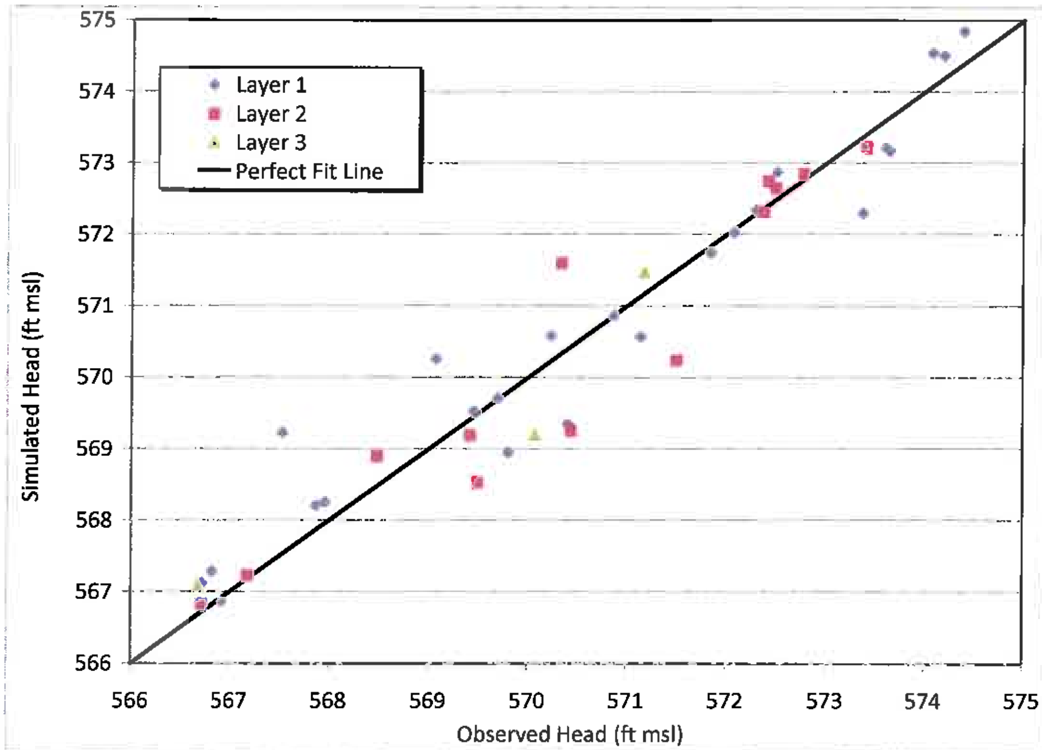
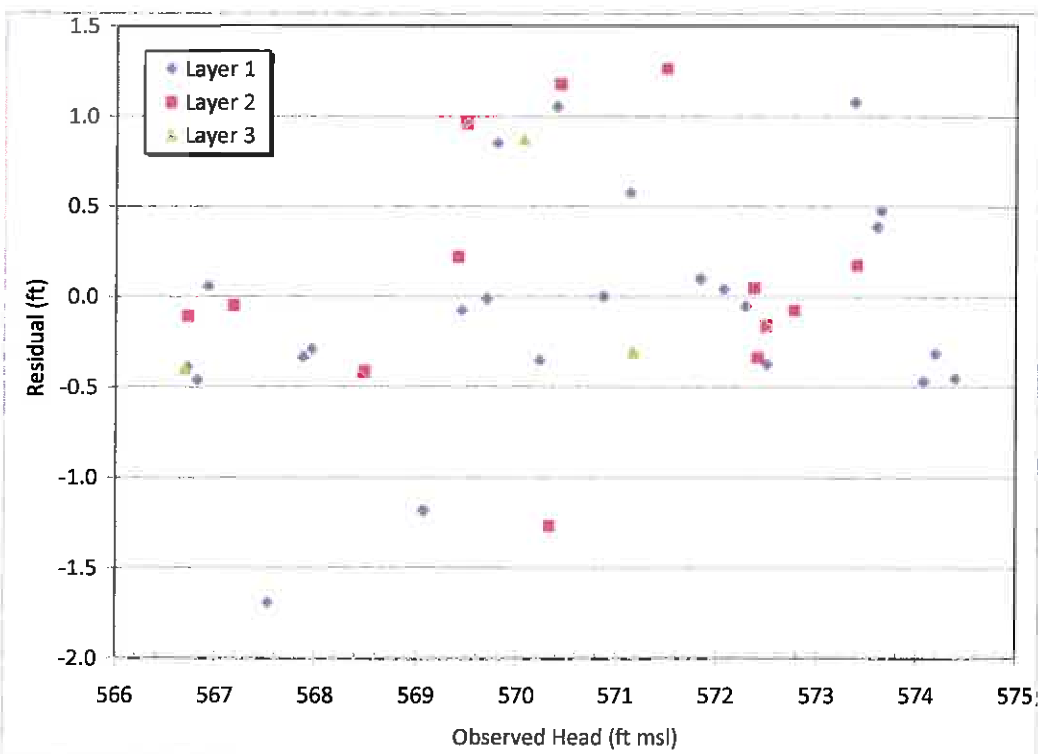


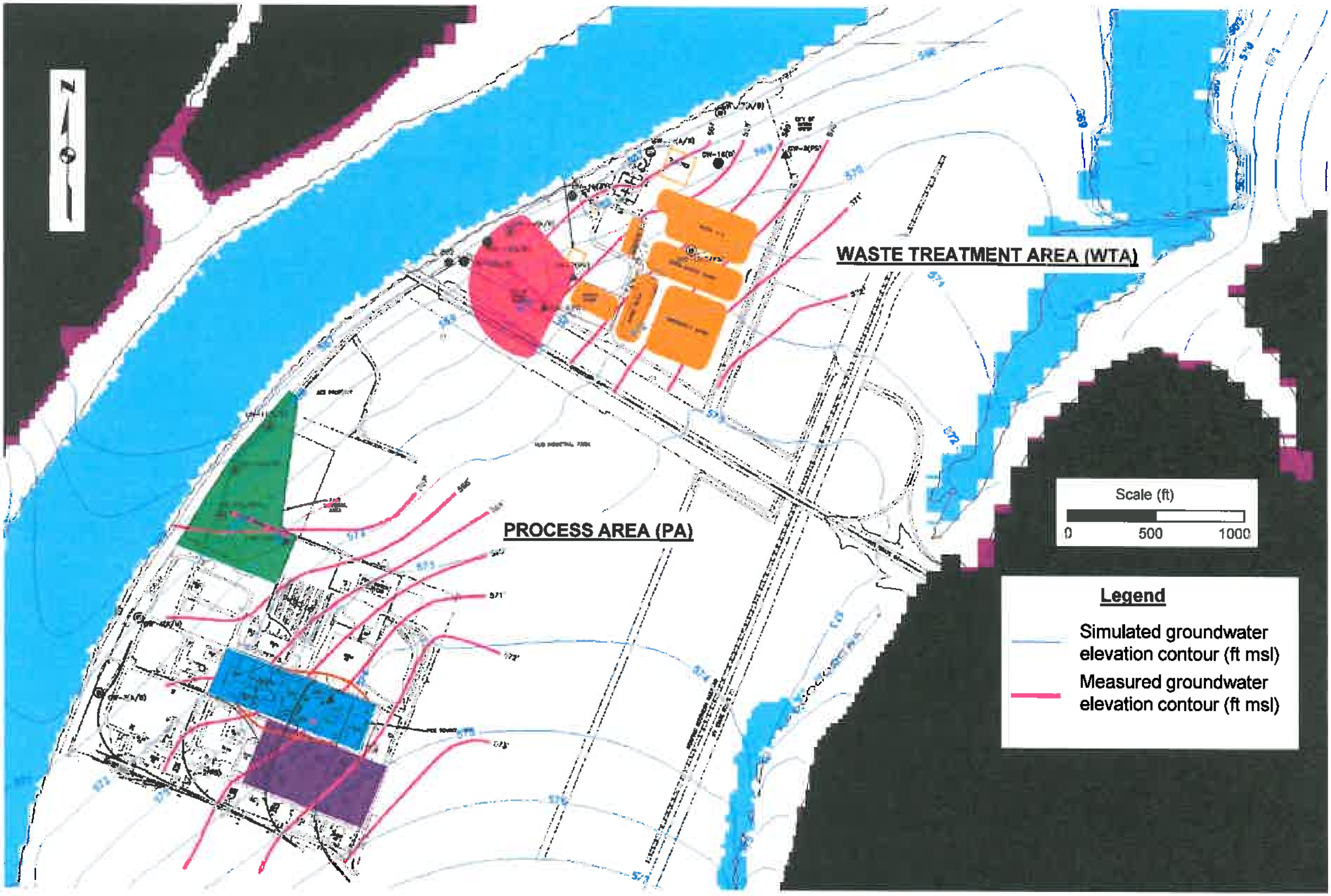
Figure 20
Calibration Plots – Residuals and Simulated Head vs. Observed Head





GSI Job No. G-3559
9 September 2011

Figure 21
Comparison of Simulated and Interpolated Potentiometric Surface in Zone A



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9 September 2011

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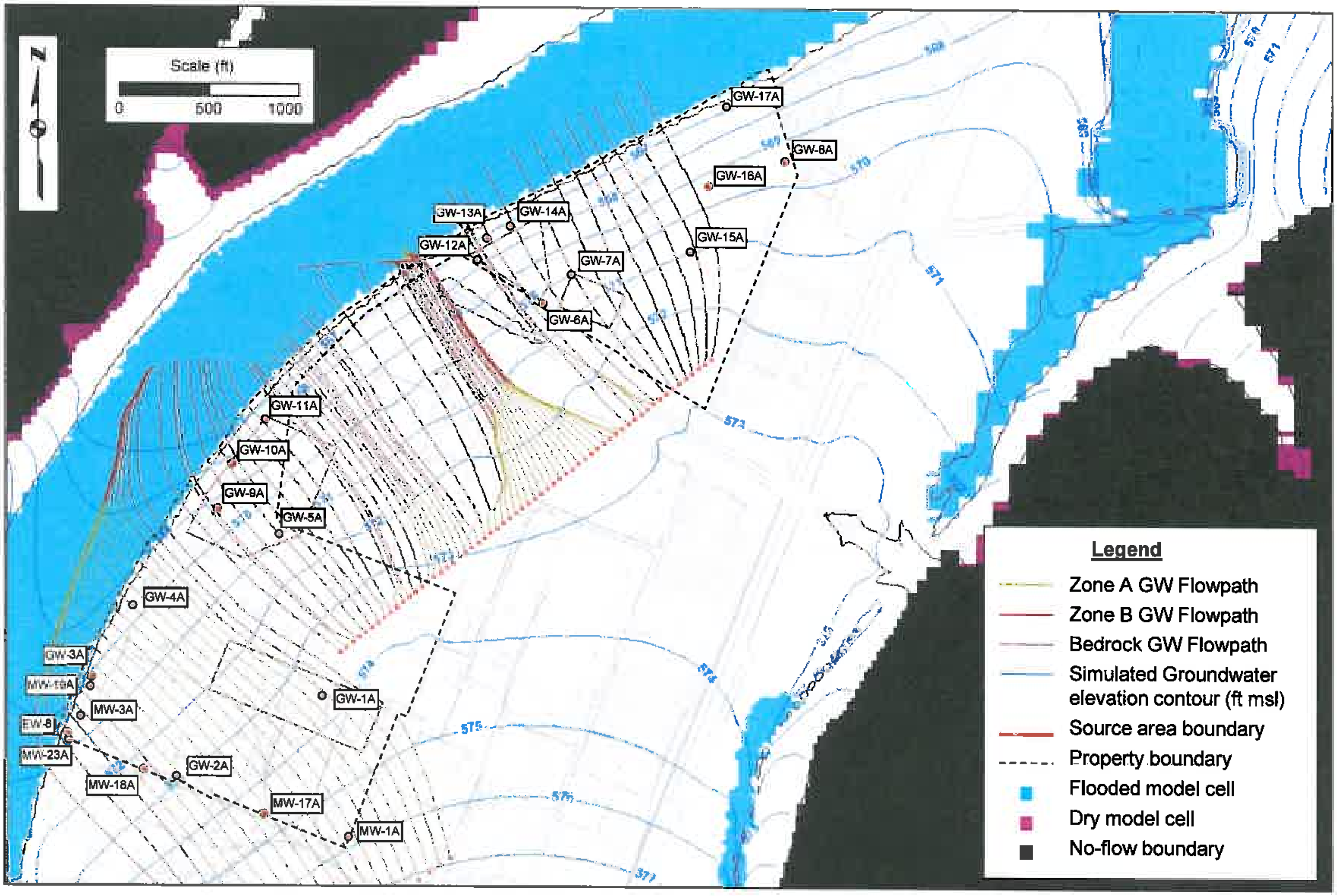
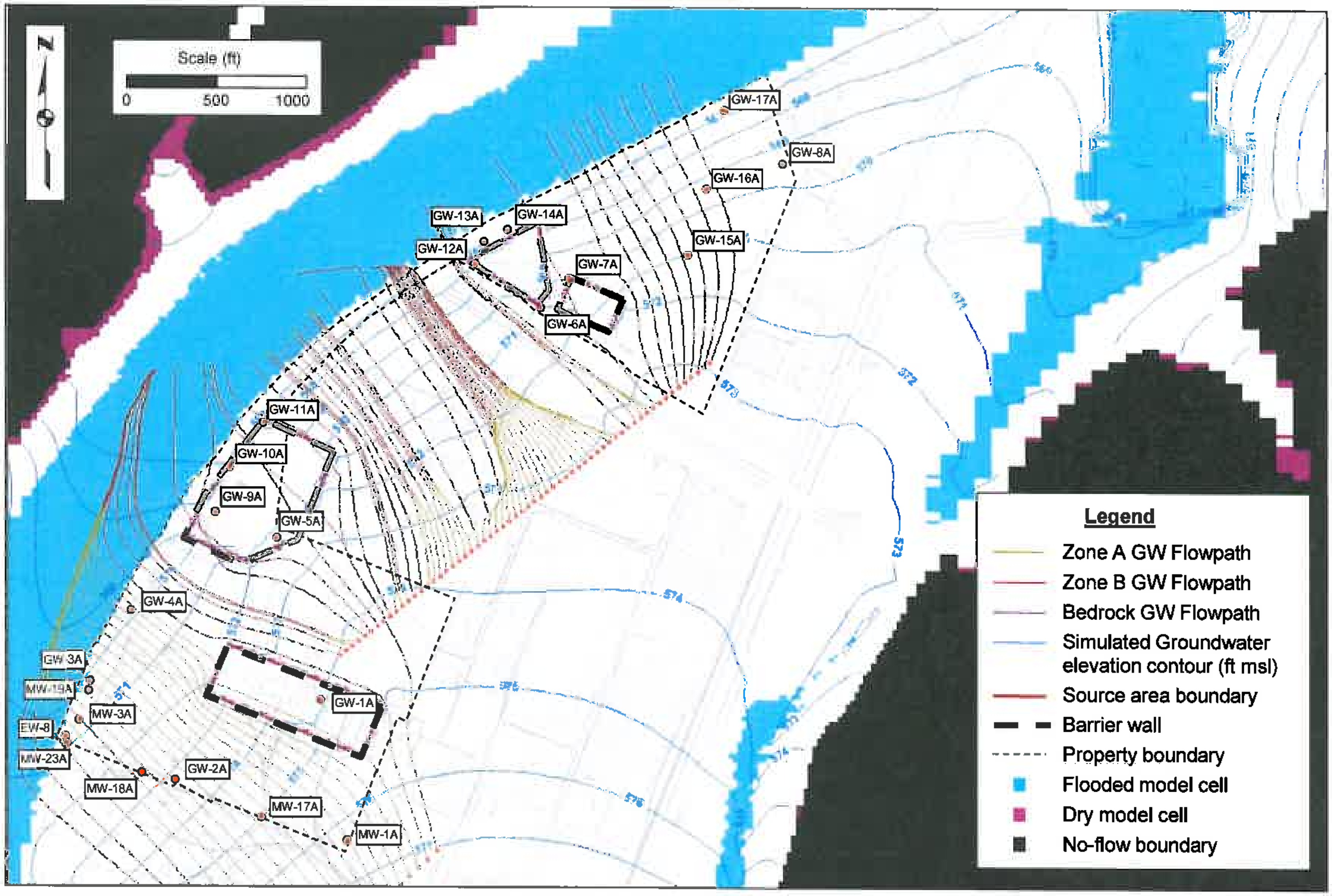
Figure 22
Simulated Flow Paths in Zone A Before Interim Measures
(Forward Particle Tracking; No Barrier Walls)



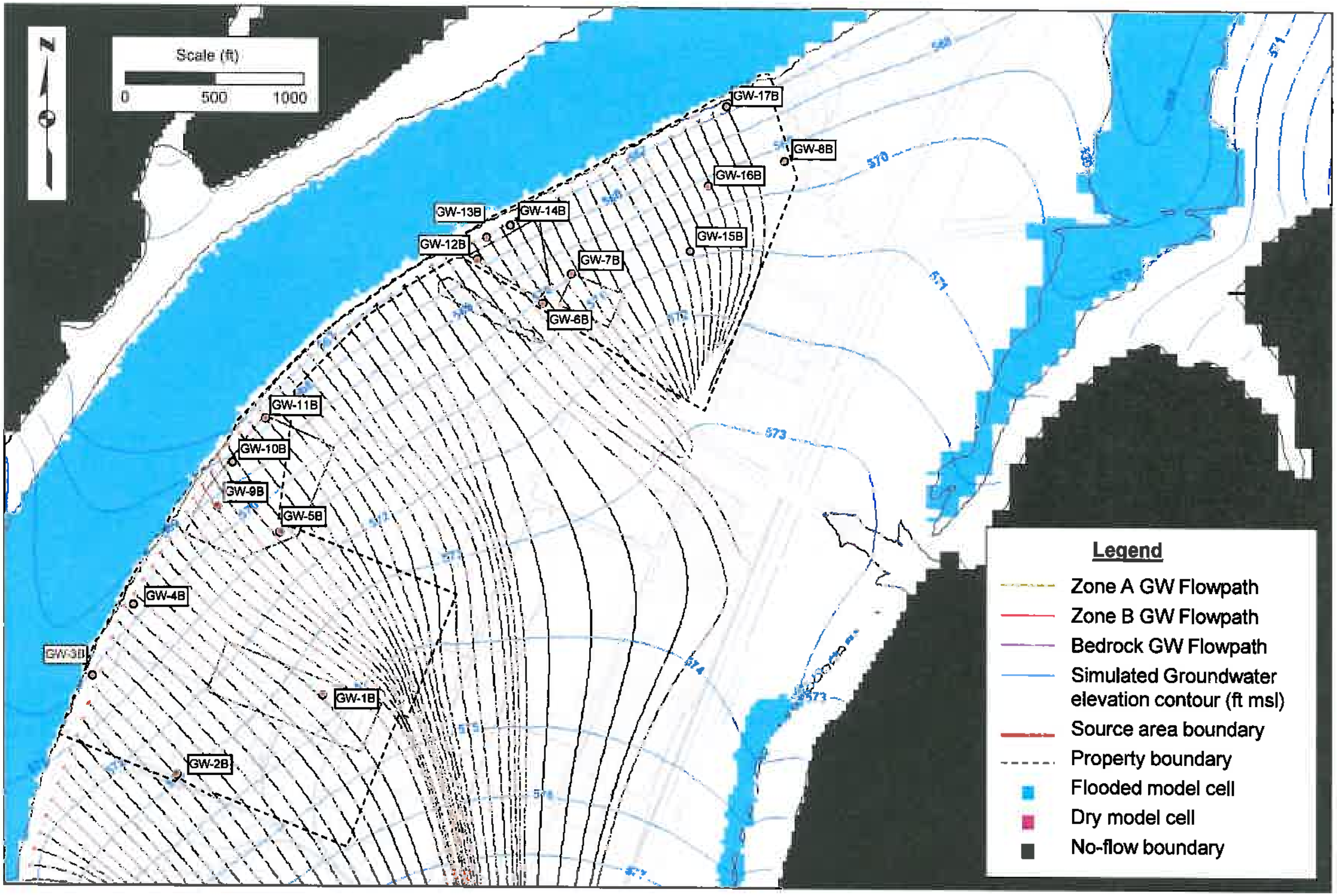
GSI ENVIRONMENTAL

Figure 23
Simulated Flow Paths in Zone A After Interim Measures
(Forward Particle Tracking; Four Barrier Walls Present)



GSI Job No. G-3559
9 September 2011

Figure 24
Simulated Flow Paths in Zone B Before Interim Measures
(Backward Particle Tracking; No Barrier Walls)



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9 September 2011

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Figure 25
Simulated Flow Paths in Zone B After Interim Measures
(Backward Particle Tracking; Four Barrier Walls Present)

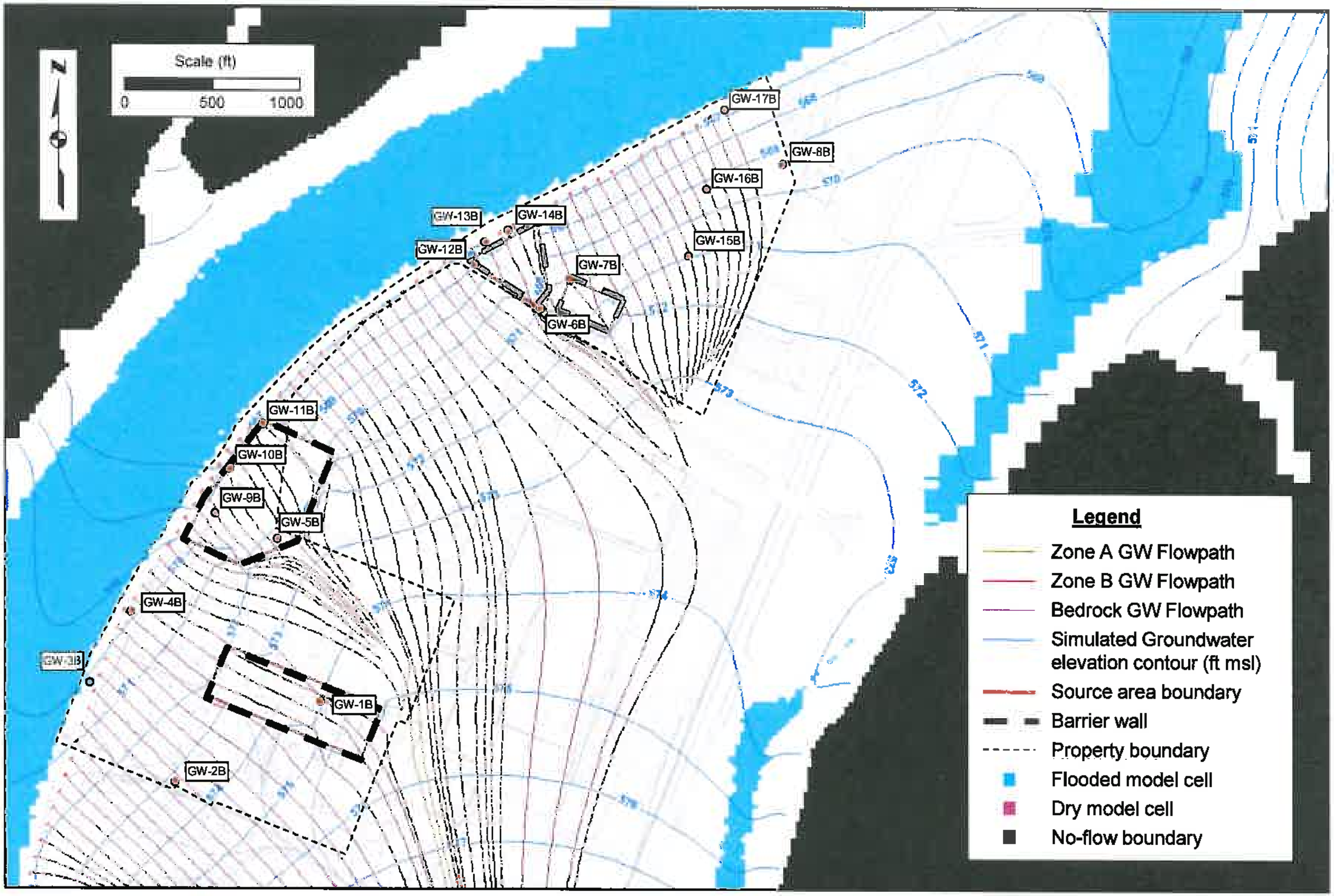


Figure 26
Change in Zone A and Zone B Heads After Barrier Wall Construction

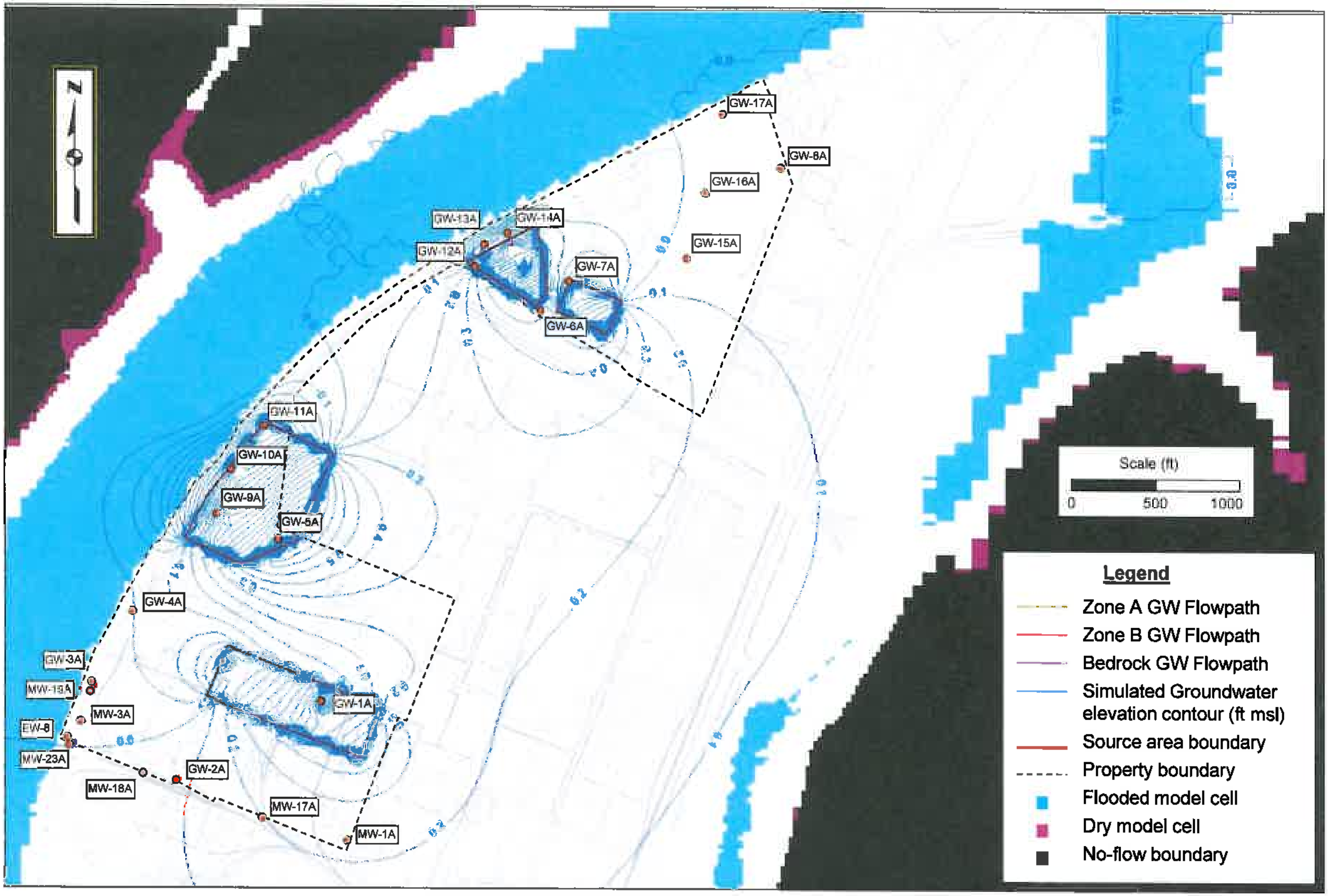
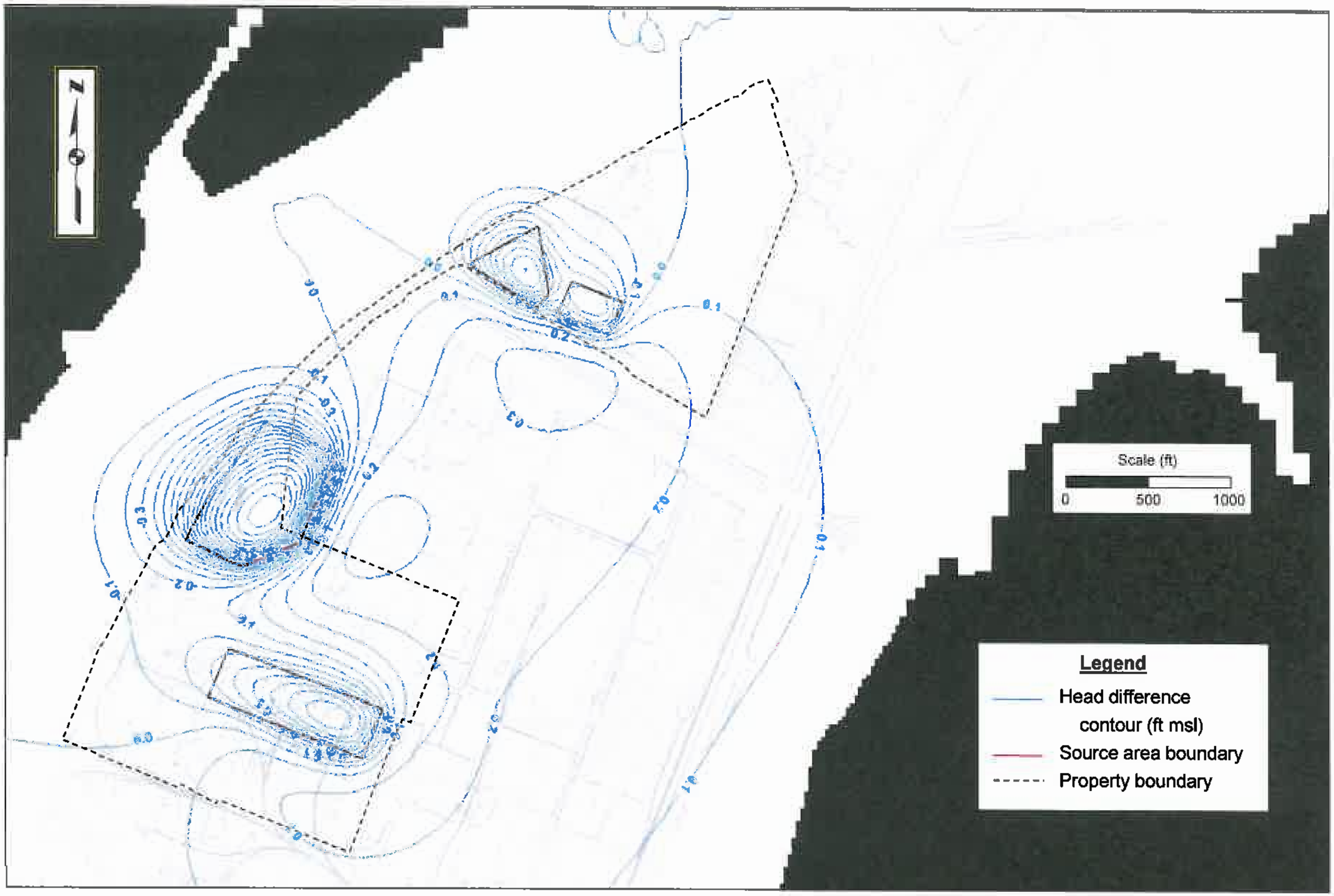
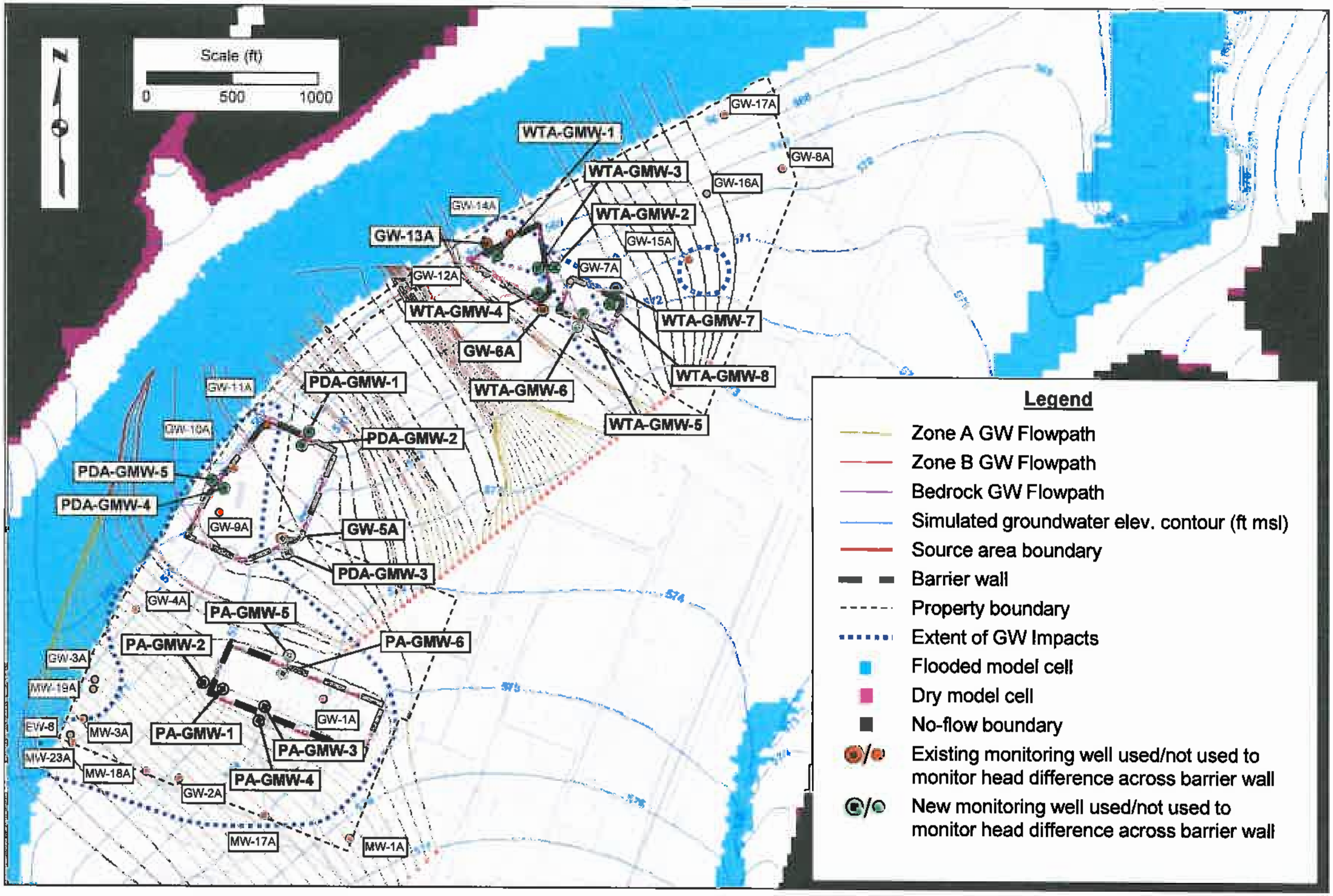


Figure 27
Change in Bedrock Heads After Barrier Wall Construction



GSI ENVIRONMENTAL

Figure 28
Proposed Changes to Monitoring Well Network in Zone A



GSI Job No. G-3559
9 September 2011

Figure 29
Proposed Changes to Monitoring Well Network in Zone B

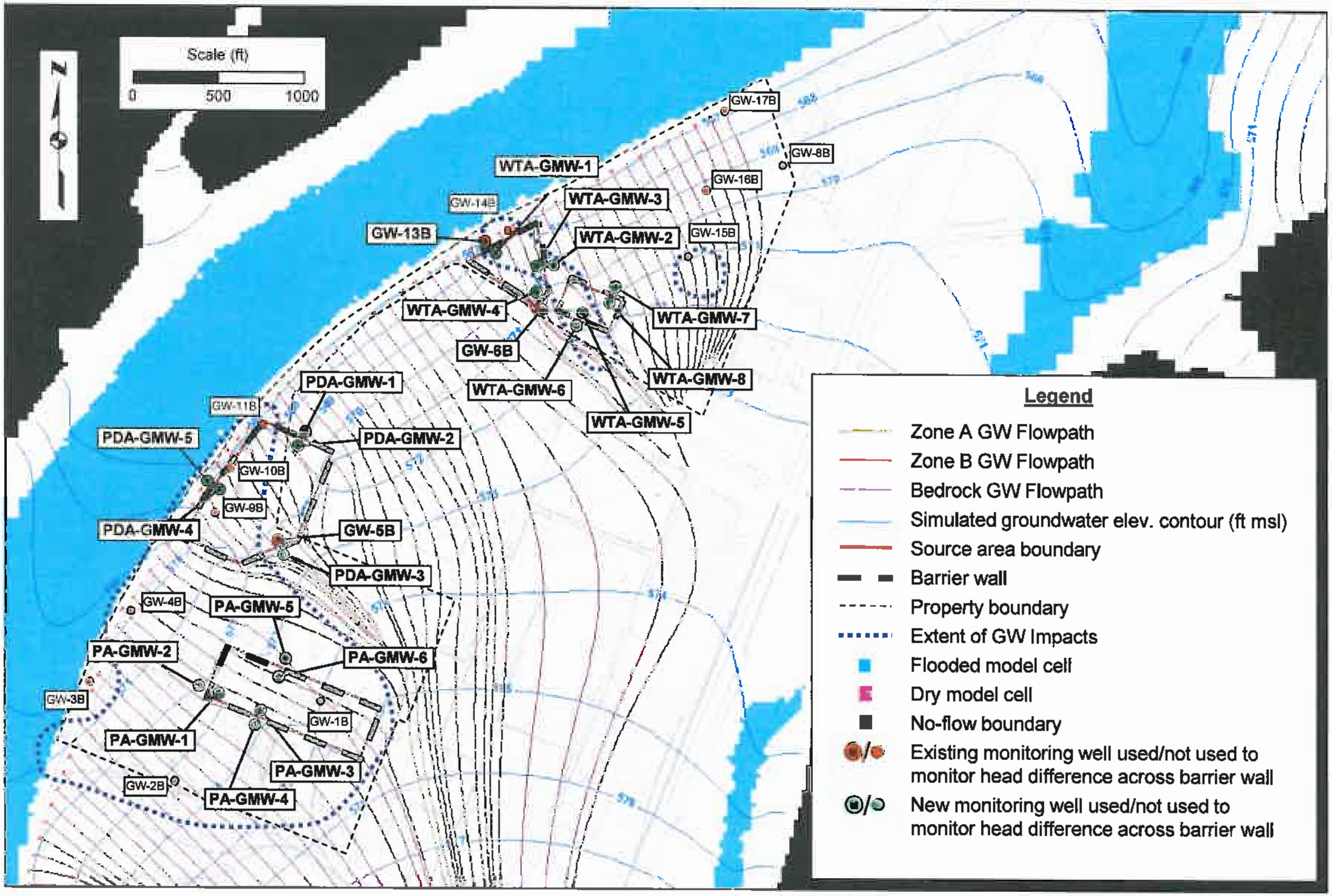
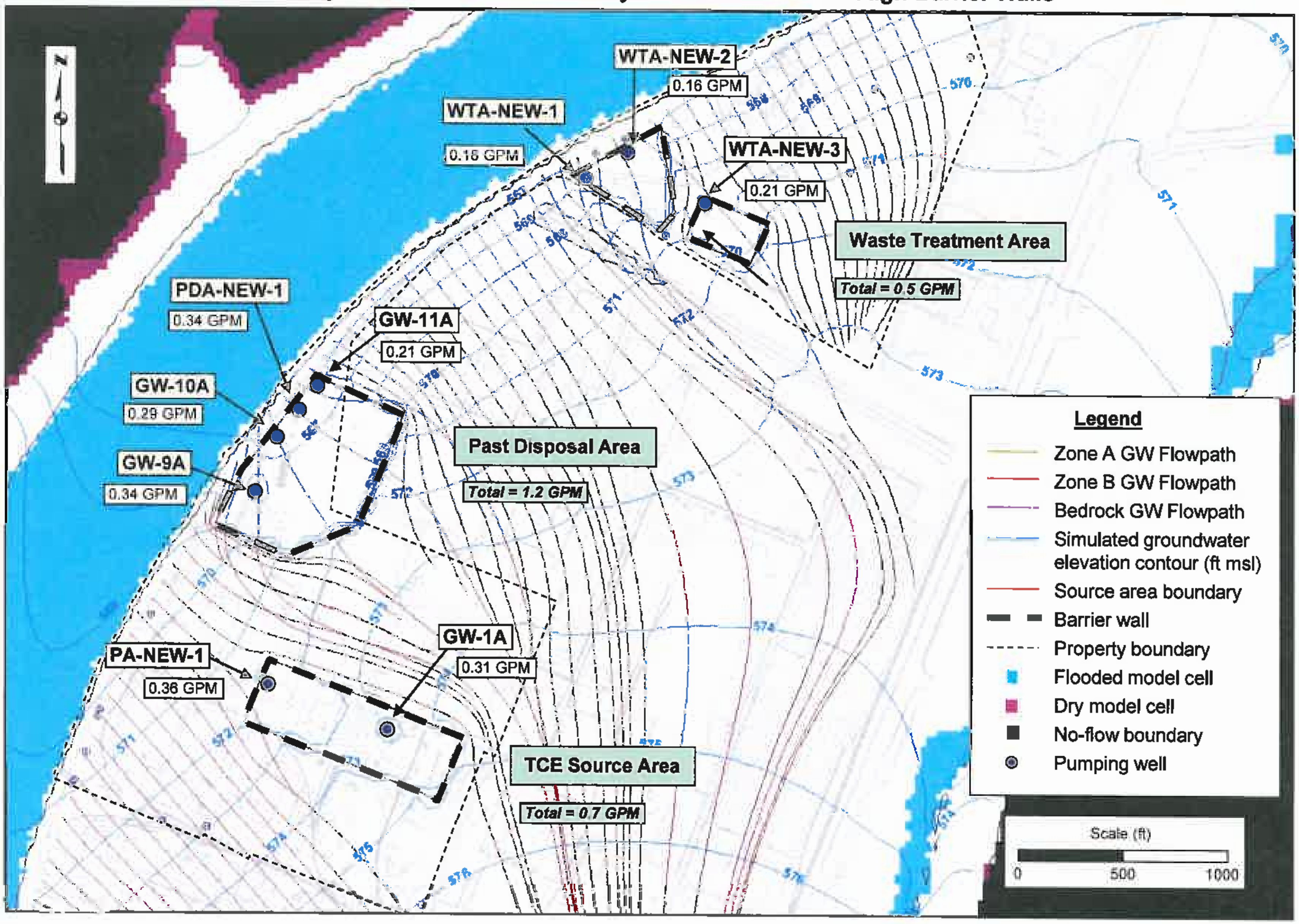


Figure 30
Well Locations and Minimum Pumping Rates
Required to Maintain Inward Hydraulic Gradient Through Barrier Walls



GSI ENVIRONMENTAL

Figure 31
Capture Zones of Pumping Wells at the WTA

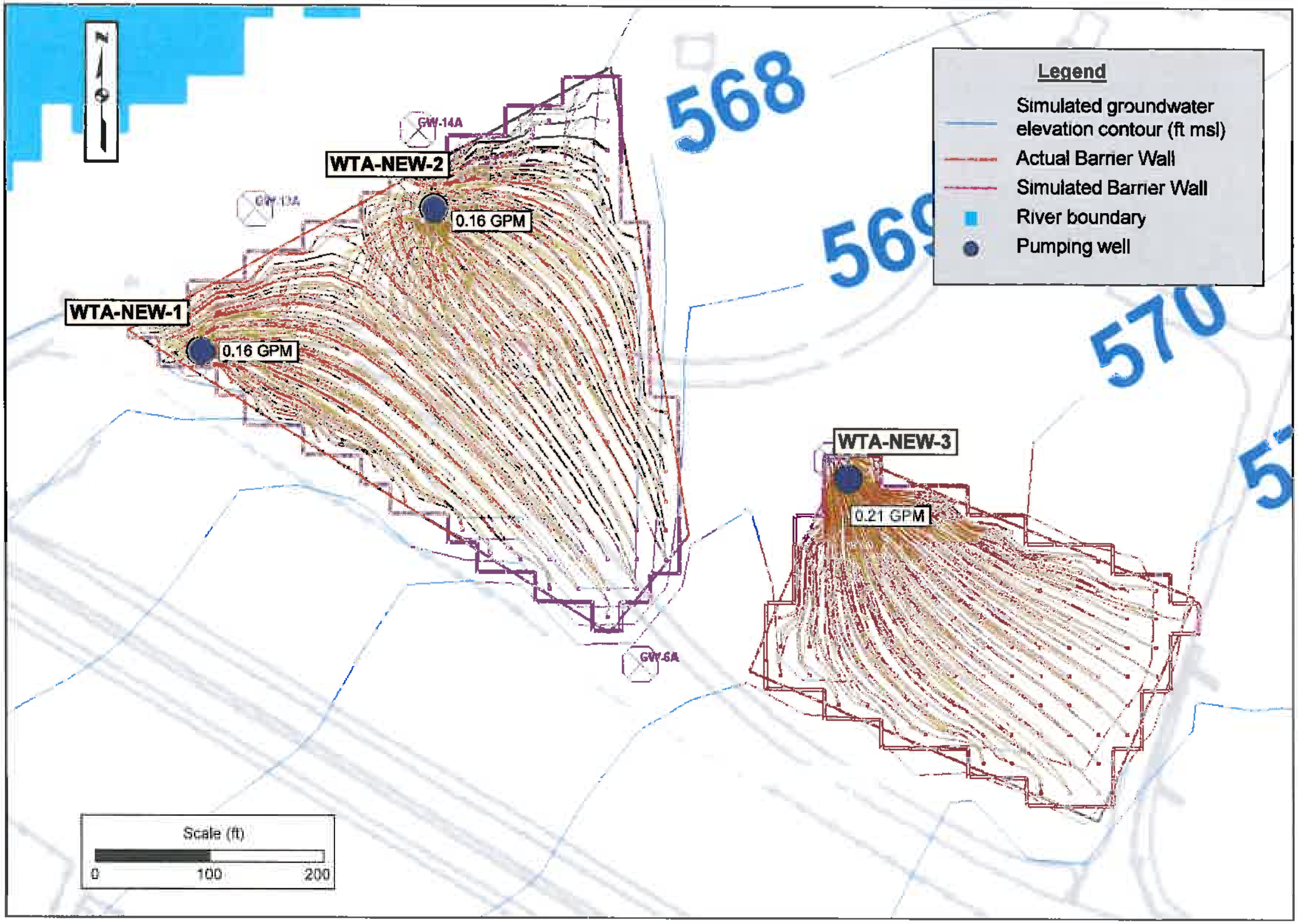


Figure 32
Capture Zones of Pumping Wells at the PDA

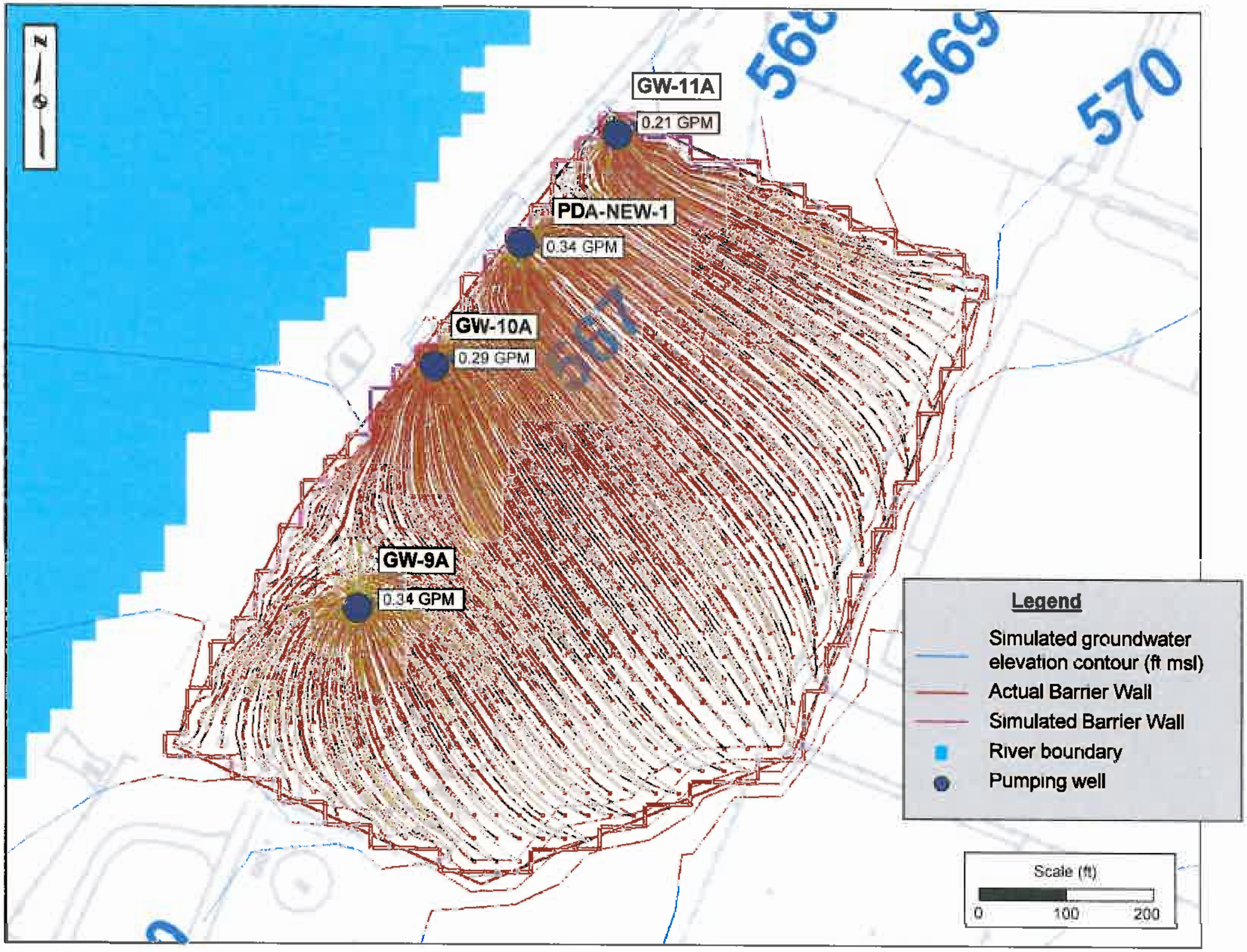


Figure 33
Capture Zones of Pumping Wells at the PA

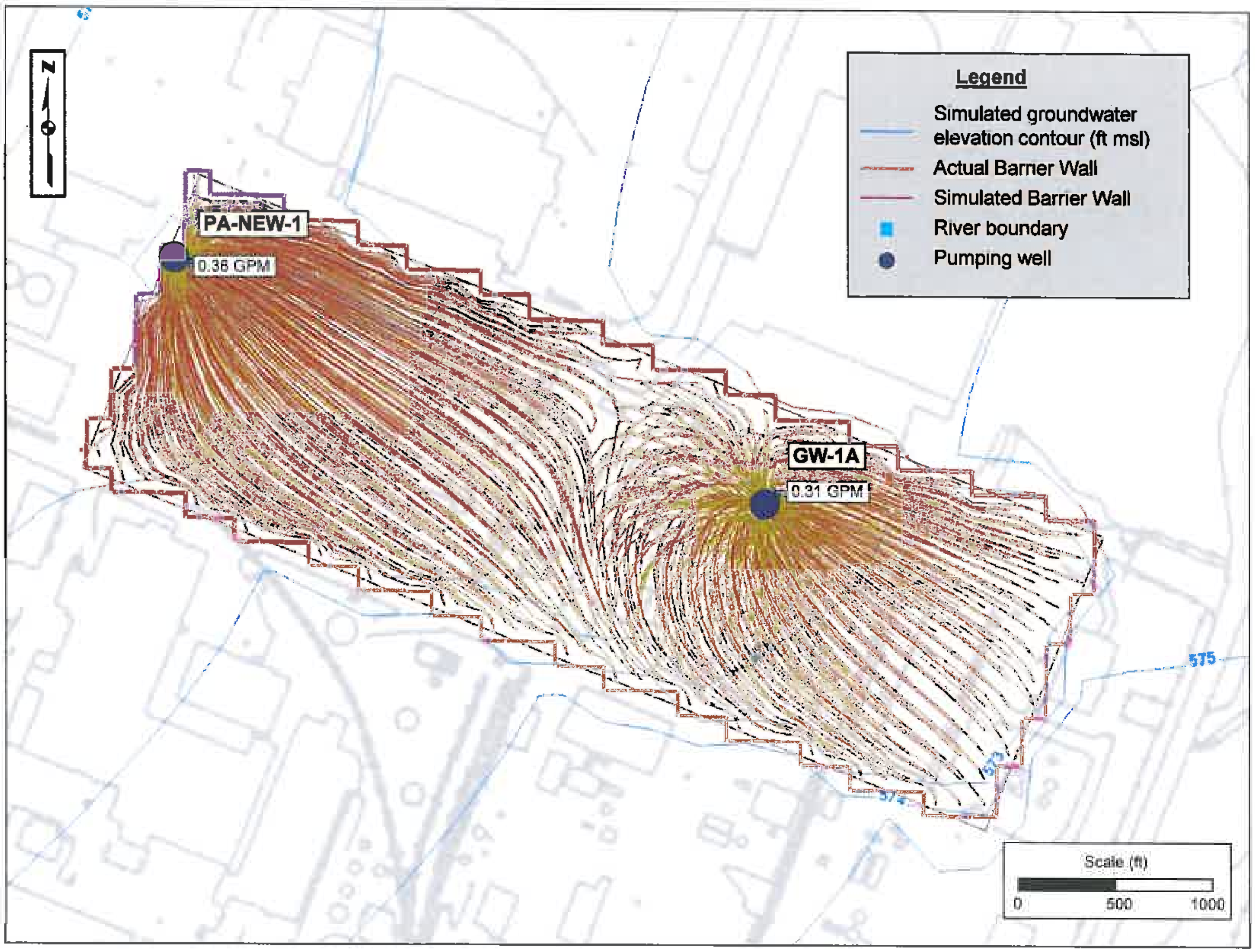


Figure 34
Percent Groundwater Flow Reduction Through Source Areas as a Function of Pumping Rate Inside Barrier Walls

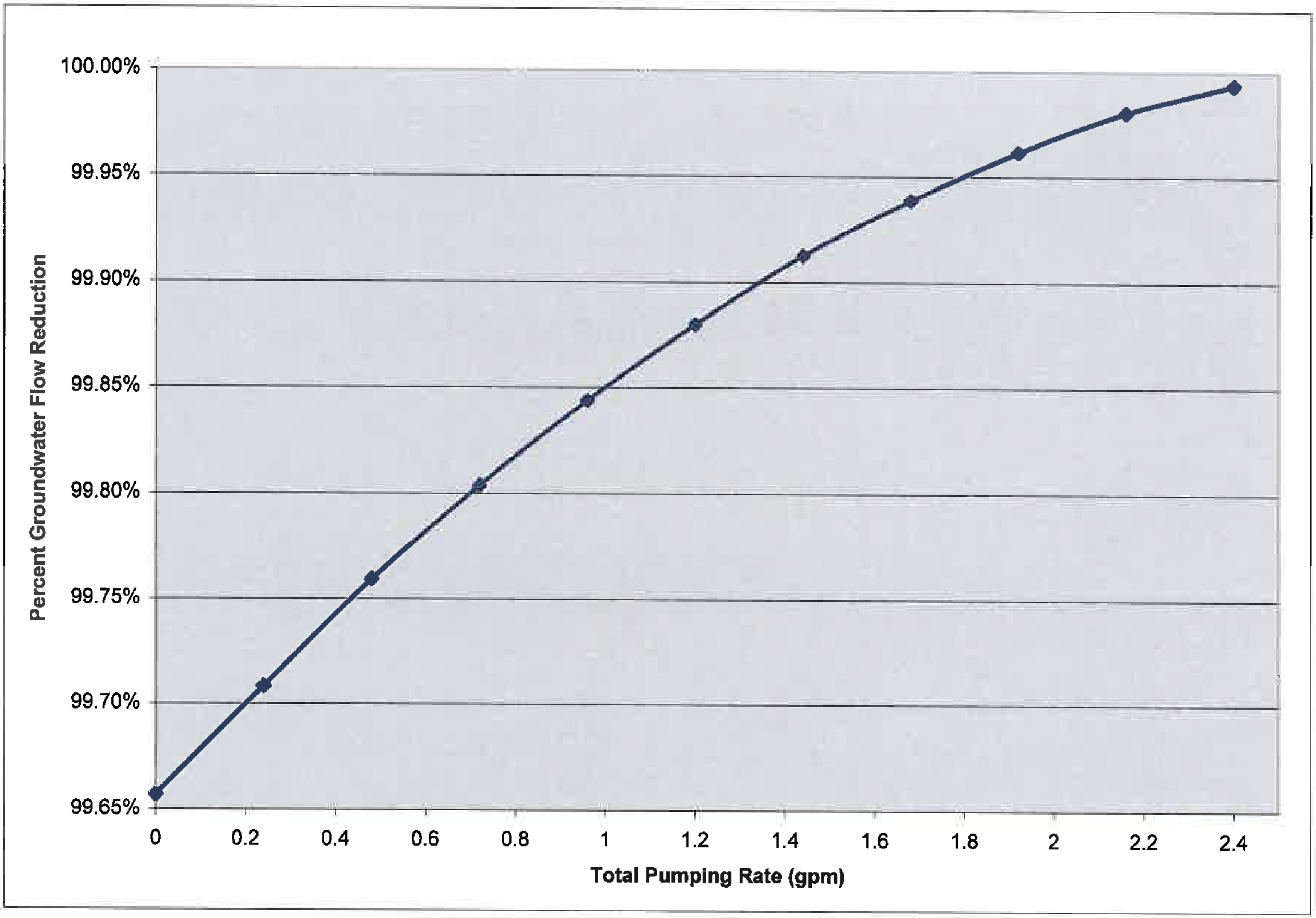
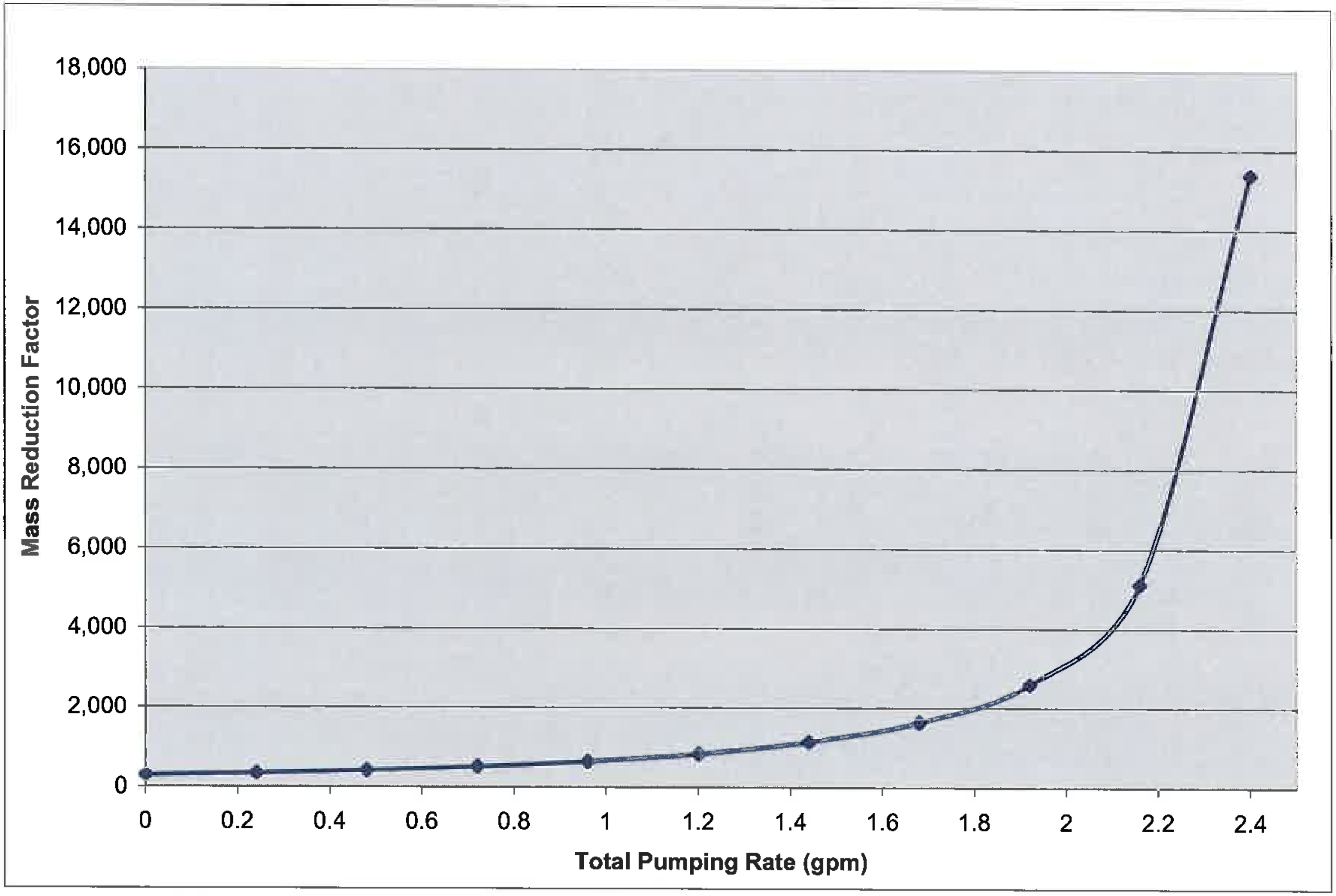
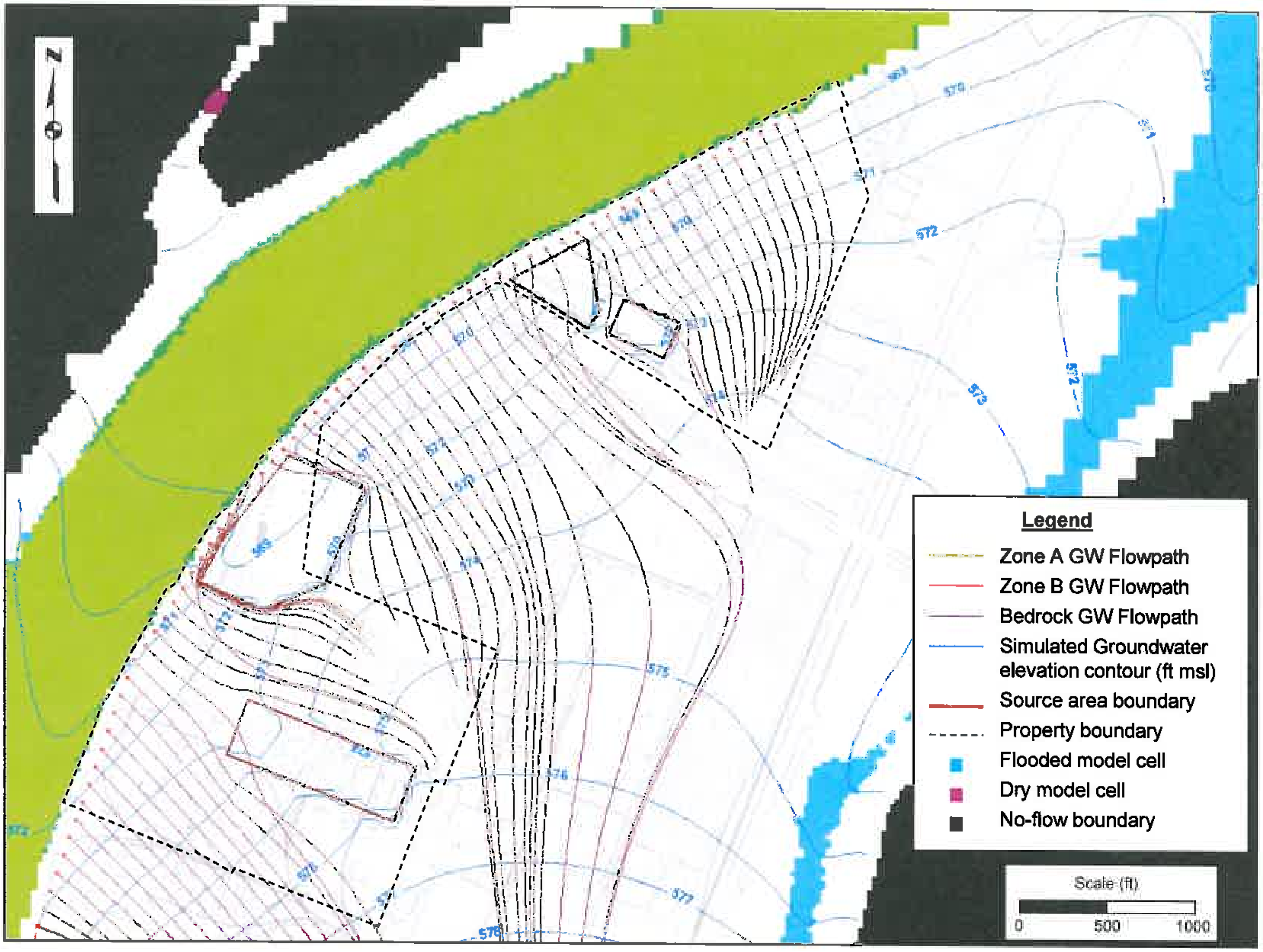


Figure 35
Mass Reduction Factor as a Function of Total Pumping Rate Inside Barrier Walls



GSI Job No. G-3559
9 September 2011

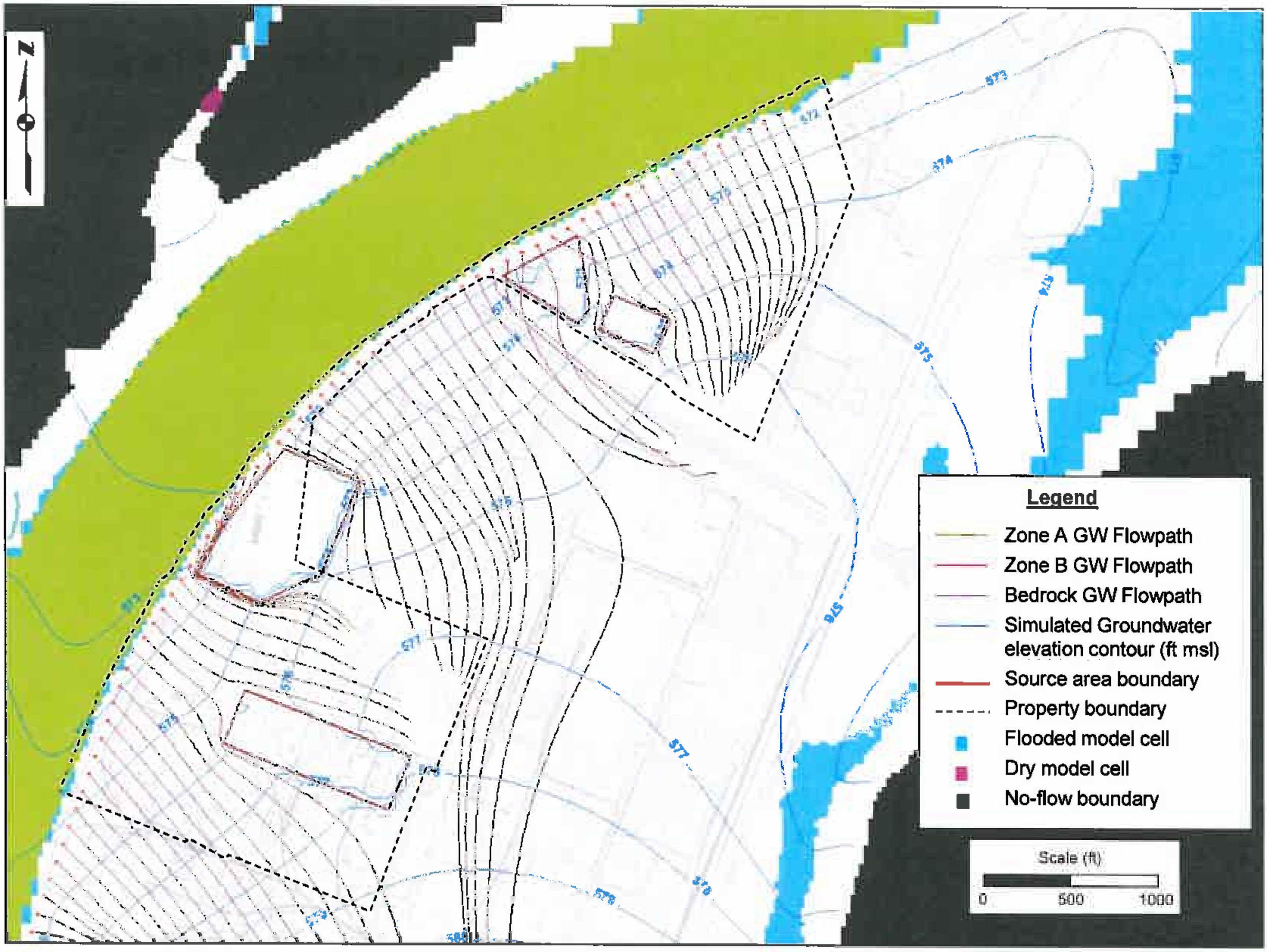
Figure 36
Simulated Zone B Potentiometric Surface and Groundwater Flow Paths with Kanawha River Elevation 2 Feet Higher (368 ft msl)



GSI Job No. G-3559
9 September 2011

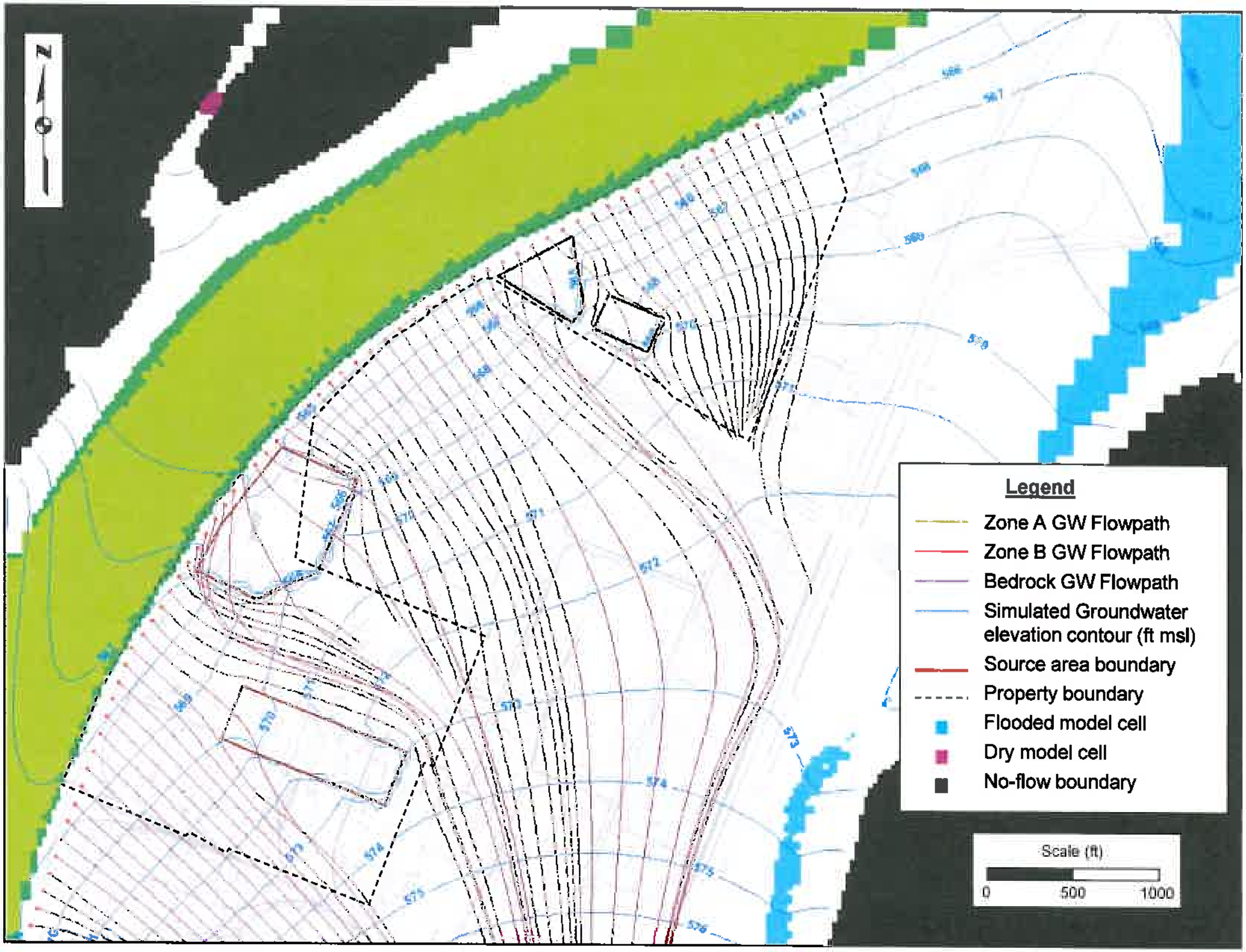
Figure 37
Simulated Zone B Potentiometric Surface and Groundwater Flow Paths with Kanawha River Elevation 5 Feet Higher (371 ft msl)

GSI ENVIRONMENTAL



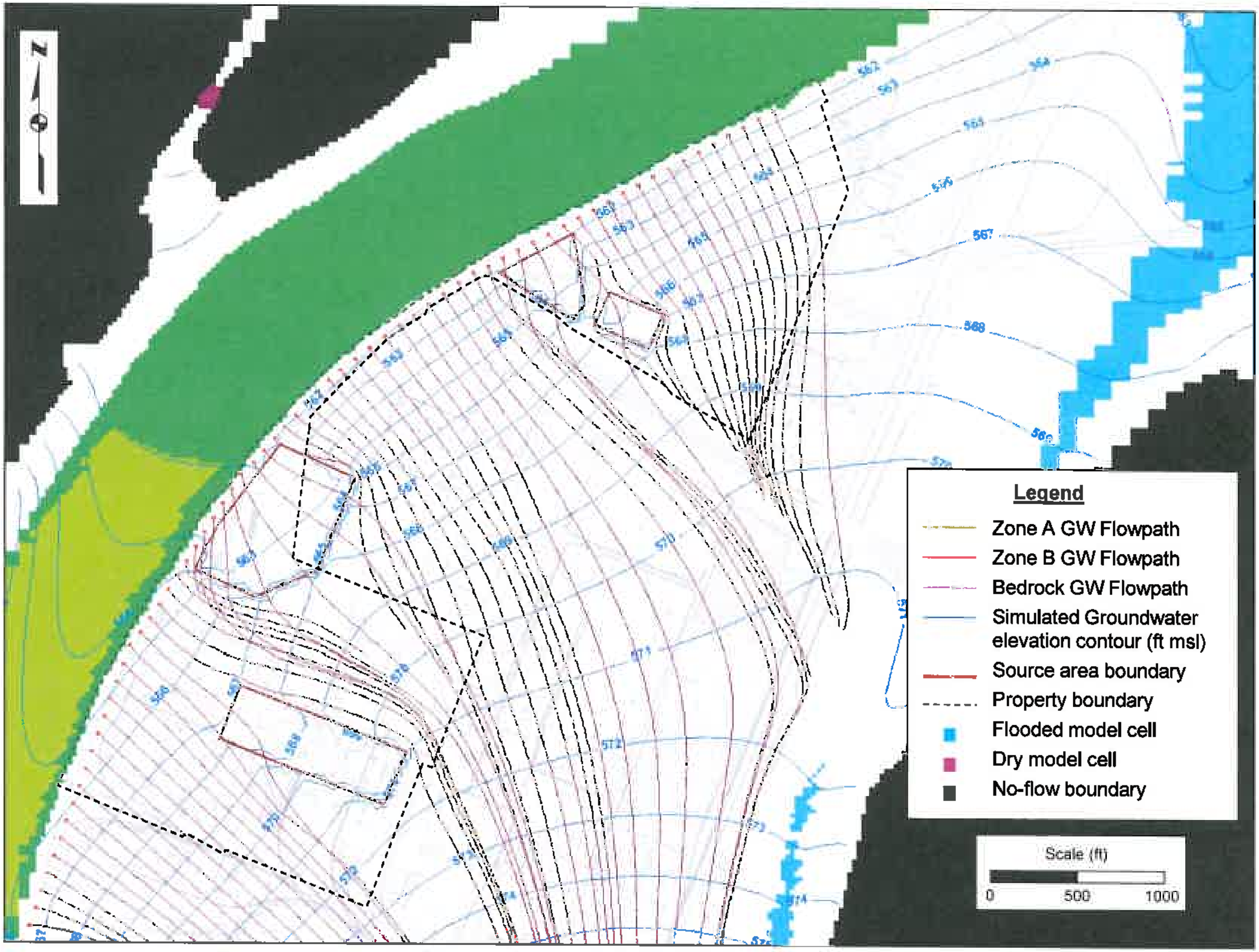
GSI Job No. G-3559
9 September 2011

Figure 38
Simulated Zone B Potentiometric Surface and Groundwater Flow Paths with Kanawha River Elevation 2 Feet Lower (364 ft msl)



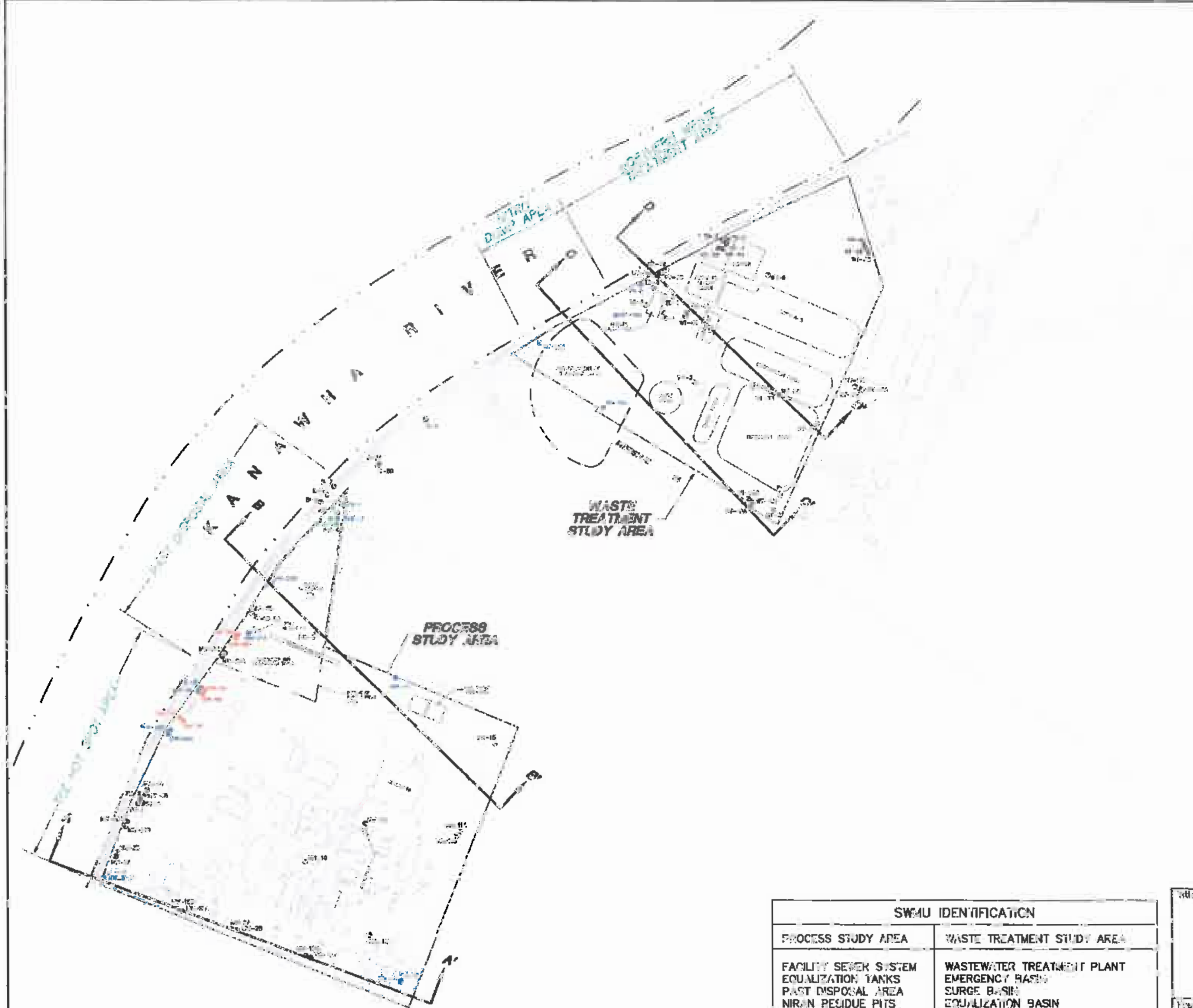
GSI Job No. G-3559
9 September 2011

Figure 39
Simulated Zone B Potentiometric Surface and Groundwater Flow Paths with Kanawha River Elevation 5 Feet Lower (361 ft msl)



APPENDICES

Appendix A
Cross-Sections Prepared by Roux and Associates (1999).



LEGEND

MONITORING WELL LOCATION AND IDENTIFICATION

MONITORING WELL LOCATION AND IDENTIFICATION FROM QUARTERLY PERFORMANCE MONITORING PROGRAM

LNAPL EXTRACTION WELL LOCATION AND IDENTIFICATION

GROUND-WATER EXTRACTION WELL LOCATION AND IDENTIFICATION

PIEZOMETER LOCATION AND IDENTIFICATION

EDGE OF WATER

FENCE

FORMER EXTENT OF NITRO DUMP AREA

A A' CROSS-SECTION LOCATION

NOTE

1.) DASHED CONTOURS EXHIBIT HEAVY FOLIAGE.

REFERENCE

1.) TOPOGRAPHIC MAP PREPARED BY [illegible] CORPORATION DATED 9/17/9[illegible]

400' 0' 400'

SWMU IDENTIFICATION	
PROCESS STUDY AREA	WASTE TREATMENT STUDY AREA
FACILITY SEWER SYSTEM EQUALIZATION TANKS PAST DISPOSAL AREA NIRAN RESIDUE PITS TEEPEE INCINERATOR BUILDING 46 INCINERATOR	WASTEWATER TREATMENT PLANT EMERGENCY BASIN SURGE BASIN EQUALIZATION BASIN LIMESTONE BED WASTE POND DECONTAMINATED 2,4,5-T BUILDING CITY OF NITRO DUMP

Title:

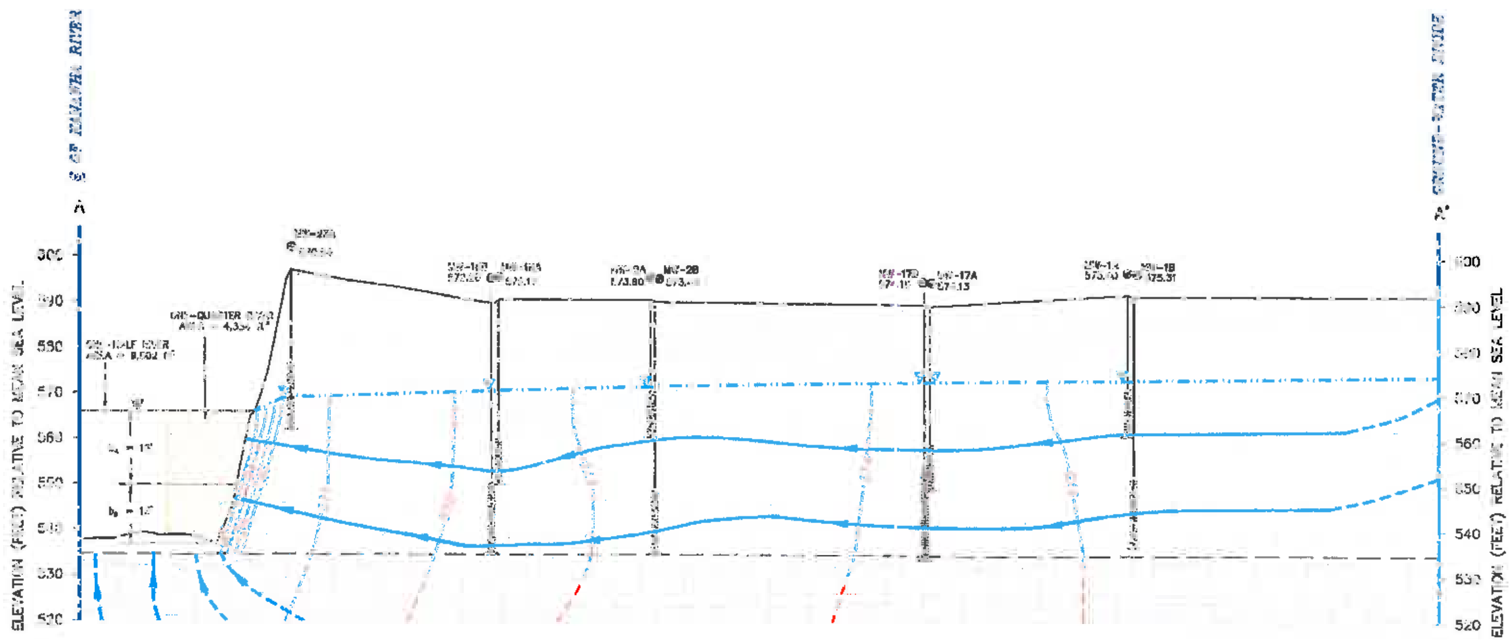
SITE PLAN

NITRO, WEST VIRGINIA

Prepared For:

ROUX ASSOCIATES INC

Compiled by:	Date:	Figure
Prepared by:	Scale:	2
Project Mgr:	Revision:	
Proj No:	File No:	



VERTICAL EXAGGERATION 10X

LEGEND

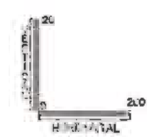
- CROSS-SECTIONAL RIVER AREA
- MW-9A — MONITORING WELL LOCATION AND IDENTIFICATION
- GROUND-WATER ELEVATION (FT)
- WELL CASING
- WELL SCREEN
- BOTTOM OF WELL
- WATER TABLE SURFACE
- SURFACE WATER ELEVATION
- GROUND-WATER FLOW DIRECTION (DASHED WHERE INFERRED)
- POTENTIOMETRIC GROUND-WATER CONTOUR (DASHED WHERE INFERRED)

NOTES :

1. GROUND-WATER ELEVATION MEASURED ON SEPTEMBER 2?, 1994.
2. KANAWHA RIVER CROSS-SECTION INFORMATION WAS GENERATED FROM THE KANAWHA RIVER SURVEY OF 1929–1930 FROM PLATES 38 AND 39 DATED JANUARY 15, 1931.
3. REFER TO PLATE 1 FOR LOCATION OF CROSS-SECTION.
4. GROUND SURFACE TOPOGRAPHY BASED UPON SURVEYED WELL ELEVATIONS.

PREDOMINANT SOIL TYPE LEGEND

FILL CONTAINING SAND, SILT, AND CLAY	FINE TO MEDIUM SAND
SILT AND CLAY	MEDIUM TO COARSE SAND
SILT AND SAND	SILTSTONE BEDROCK



Title:

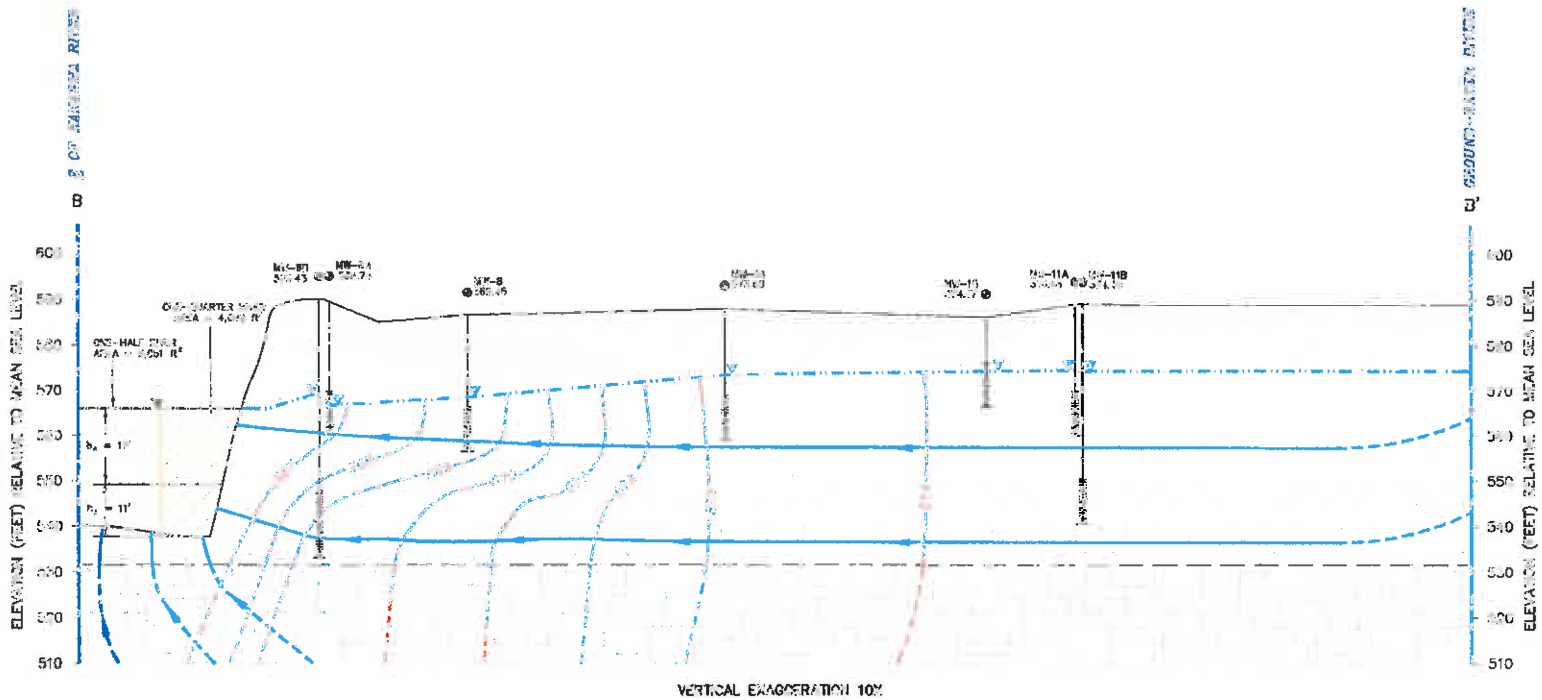
GROUND–WATER FLOW REPRESENTATION CROSS–SECTION A – A' TCE HOT SPOT AREA

NITRO, WEST VIRGINIA

Prepared For:

ROUX ASSOCIATES INC

Compiled by: J.R.G.	Date: [illegible]	Figure
Prepared by: J.R.R.	Scale: SHOWN	3
Project Mgr: P.J.P.	Revision:	
Project No: [illegible]	File No: [illegible]143	



LEGEND

CROSS-SECTIONAL SWALE AREA

MONITORING WELL LOCATION AND IDENTIFICATION

GROUND-WATER ELEVATION (FT)

WELL CASING

WELL SCREEN

BOTTOM OF WELL

WATER TABLE SURFACE

SURFACE WATER ELEVATION

GROUND-WATER FLOW DIRECTION (DASHED WHERE INFERRED)

POTENTIOMETRIC GROUND-WATER CONTOUR (DASHED WHERE INFERRED)

NOTES :

1. GROUND-WATER ELEVATION MEASURED ON SEPTEMBER 20, 1994.
2. KANAWHA RIVER CROSS-SECTION INFORMATION WAS GENERATED FROM THE KANAWHA RIVER SURVEY OF 1929-1930 FROM PLATES 38 AND 39 DATED JANUARY 15, 1931.
3. REFER TO PLATE 1 FOR LOCATION OF CROSS-SECTION.
4. GROUND SURFACE TOPOGRAPHY BASED UPON SURVEYED WELL ELEVATIONS.

PREDOMINANT SOIL TYPE LEGEND

FILL CONTAINING SAND, SILT, AND CLAY

FINE TO MEDIUM SAND

SILT AND CLAY

MEDIUM TO COARSE SAND

SILT AND SAND

Title

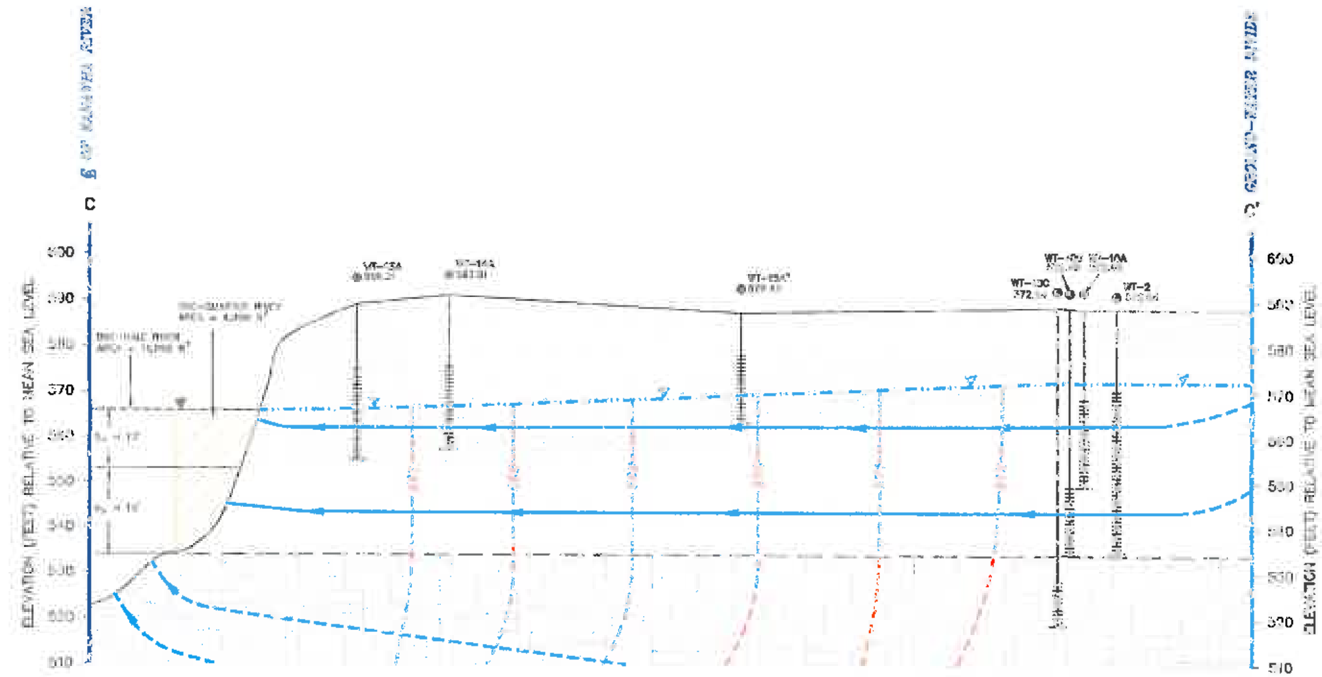
GROUND-WATER FLOW REPRESENTATION CROSS SECTION B – B' PAST DISPOSAL AREA

NITRO, WEST VIRGINIA

Prepared For:

ROUX ASSOCIATES INC

Compiled by:	Date:	Figure
Prepared by: J.L.G.	Scale: SHOWN	4
Project Mgr: R.J.T.	Revision:	
Proj. No.:	File No.:	



LEGEND

- CROSS-SECTIONAL RIVER AREA
- MT-13A — MONITORING WELL LOCATION AND IDENTIFICATION
- GROUND-WATER ELEVATION (FT)
- * ANOMALOUS WATER-LEVEL ELEVATION DUE TO PERCHED WATER CAUSED BY UNDERLYING LOCALIZED DENSE CLAY (NOT USED FOR PLOTTING)
- WELL CASING
- WELL SCREEN
- BOTTOM OF WELL
- WATER TABLE SURFACE
- SURFACE WATER ELEVATION
- GROUND-WATER FLOW DIRECTION (DASHED WHERE INFERRED)
- POTENTIOMETRIC GROUND-WATER CONTOUR (DASHED WHERE INFERRED)

NOTES :

1. GROUND-WATER ELEVATION MEASURED ON 9/20/94.
2. KANAWHA RIVER CROSS-SECTION INFORMATION WAS GENERATED FROM THE KANAWHA RIVER SURVEY OF 1929-1930 FROM PLATES 38 AND 39 DATED JANUARY 15, 1931.
3. REFER TO PLATE 1 FOR LOCATION OF CROSS SECTION.
4. GROUND SURFACE TOPOGRAPHY BASED UPON SURVEYED WELL ELEVATIONS.

PREDOMINANT SOIL TYPE LEGEND

- FILL CONTAINING SAND, SILT, AND CLAY
- SILT AND CLAY
- SILT AND SAND
- FINE TO MEDIUM SAND
- MEDIUM TO COARSE SAND
- SILTSTONE BEDROCK

HORIZONTAL

Title:

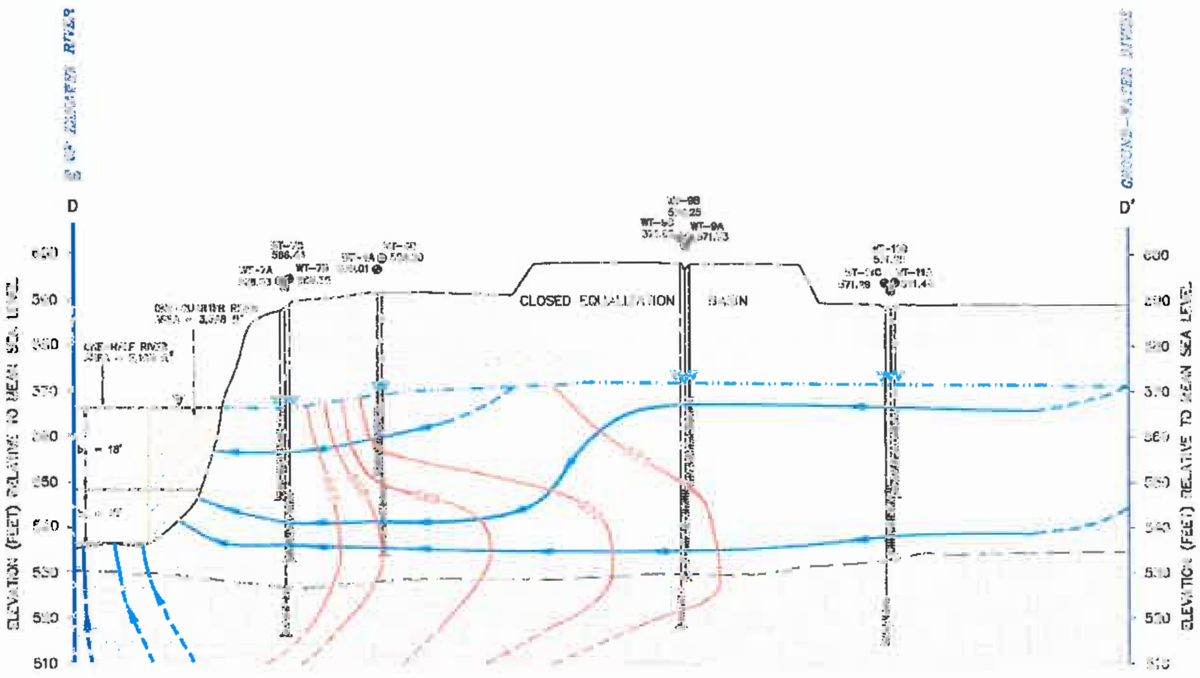
GROUND-WATER FLOW REPRESENTATION CROSS-SECTION C - C'
NITRO DUMP AREA
NITRO, WEST VIRGINIA

Prepared For:

[redacted]

ROUX ASSOCIATES INC

Compiled by:	Date:	Figure
Prepared by:	Scale: SHOWN	5
Project Mgr: P.J.P.	Revision:	
Proj No:	File No:	



VERTICAL EXAGGERATION 10X

LEGEND

- CROSS-SECTIONAL RIVER AREA
- MONITORING WELL LOCATION AND IDENTIFICATION
- 570.01 GROUND-WATER ELEVATION (FT)
- WELL CASING
- WELL SCREEN
- BOTTOM OF WELL
- WATER TABLE SURFACE
- SURFACE WATER ELEVATION
- GROUND-WATER FLOW DIRECTION (DASHED WHERE INFERRED)
- POTENTIOMETRIC GROUND-WATER CONTOUR (DASHED WHERE INFERRED)

NOTES :

1. GROUND-WATER ELEVATION MEASURED ON SEPTEMBER 25, 1994.
2. KANAWHA RIVER CROSS-SECTION INFORMATION WAS GENERATED FROM THE KANAWHA RIVER SURVEY OF 1929-1930 FROM PLATES 29 AND 30 DATED JANUARY 15, 1931.
3. REFER TO PLATE 1 FOR LOCATION OF CROSS-SECTION.
4. GROUND SURFACE TOPOGRAPHY BASED UPON SURVEYED WELL ELEVATIONS.

PREDOMINANT SOIL TYPE LEGEND

- FILL CONTAINING SAND, SILT, AND CLAY
- SILT AND CLAY
- SILT AND SAND
- FINE TO MEDIUM SAND
- MEDIUM TO COARSE SAND
- SILTSTONE BEDROCK

VERTICAL 20
HORIZONTAL 200

GROUND-WATER FLOW REPRESENTATION
CROSS SECTION D – D'
NORTHERN WASTE TREATMENT AREA

NITRO, WEST VIRGINIA

Prepared for:

Figure 8